NUMERICAL MODEL FOR AGGRADATION AND DEGRADATION IN TREE-TYPE CHANNEL NETWORKS

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Happiness lies in the joy of achievement and the thrill of creative effort.

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"The most powerful force on earth is love". This work dedicated to my parents and my brother without whose love I would not have reached this stage.

Certificate

It is certified that this work entitled **Numerical Model For Aggradation And Degradation In Tree-type Channel Networks** by *H. Rajagopal* has been carried out under my supervision and that this work has not been submitted elsewhere for a degree.

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Abstract

This work presents a numerical model for aggradation and degradation in tree-type channel networks. The model proposed numerically solves the differential equations for water flow and sediment flow taking the quasi-steady flow assumption and using uncoupled technique. The model is used to simulate aggradation and degradation profiles in converging and diverging tree-type networks. Both rectangular and non-rectangular cross-sections are considered. Three different procedures for changing the cross-sectional shape of a channel are adopted. It also includes (1) Power law model, (2) Iowa model and, (3) Meyer-Peter-Muller equations for computing the sediment transport capacity. Applicability of the model is demonstrated through several simulation studies for (1) aggradation due to sediment overloading and, (2) degradation due to sediment depletion. The numerical results shows that the proposed model satisfactorily simulates the basic features of aggradation and degradation process.

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Chapter 1

Introduction

1.1 Overview

Most natural beds are composed of loose materials which can move due to the bed shear stress exerted by the flowing water. The natural long-term (geological time scales) evolution of these channels known as alluvial channels, produces slopes, widths, depths and velocities such that the flows are able to transport the imposed sediment discharges. The amount of sediment coming into a particular reach of the channel is equal to the sediment going out from that reach and therefore, the stream bed elevation will not change over a long period of time. Such alluvial streams are said to be in regime or in equilibrium. Any short term, of order of hours to weeks, changes in flow and sediment transport rates from long-term values are accommodated by the channel by adjusting its depth, velocity and the roughness. On an intermediate time scale, of the order of months to years, channels can also adjust their large-scale geometry i.e, widths, depths, bed levels and plan form, in response to natural and, or human interference in their hydrologic and sedimentary regimes.

Many times, alluvial channels are harnessed for power generation, navigation,

and the interdependencies among them in river mechanics are too complex to permit simple analytical solutions. River response to the structural modifications extends over long distances and over many years. Therefore, evolution of this response using physical model studies is not feasible. The only alternative left is the use of mathematical models, however rudimentary they may be. The objective of the study reported herein is to develop a numerical model for aggradation and degradation of tree-type channel systems whose equillibria have been disturbed. Literature in the are of movable bed modeling is reviewed in the following section to place the present work in perspective.

1.2 Literature Review

Analytical solutions have been developed of simplified governing equations describing complex phenomenon of agradation and degradation processes. De Vries (1973) suggested that the flow can be considered quasi steady for the study of bed level variation. A linear parabolic equation for the bed level variation was obtained by assuming the flow to be uniform, and steady in a wide rectangular channel. Soni et al. (1980) used this model to predict the transient bed profiles due to sediment overloading. Jain (1981) pointed out an error in their boundary conditions and presented an analytical solution utilizing more appropriate boundary conditions. His computed results compared satisfactorily with the experimental data. Begin et al. (1981) used a similar model to compute longitudinal profiles produced by base levels lowering. Gill (1983a, 1983b) solved the linear diffusion equation for agradation and degradation by Fourier series and by the error function methods. Jaramillo and Jain (1984) developed a non-linear parabolic partial differential equation and solved it by the method of weighted residuals. They compared their computed results with experimental data obtained by Newton (1951) and Soni et al. (1980). Zhang

and Kahawita (1987) detected an error in the solution of Jaramillo and Jain (1984) and presented a numerical solution of the non-linear parabolic model. Gill (1987) also presented non-linear solutions for aggradation and degradation . Ribberink et al. (1987) suggested that a non-linear hyperbolic model should be preferred to a non-linear parabolic model.

Analytical models for aggradation and degradation are based on the assumption of steady water flow. This assumption is not valid while estimating the bed level changes during unsteady water flow conditions. It is also strictly not valid even if the discharge is constant since the water surface profile computations depend upon the bed slope. Analytical models assume also a simplified equation for sediment transport capacity and a wide rectangular cross section. Therefore, water flow equations along with sediment continuity equation are generally solved by numerical techniques. Since the early work of Vreugdenhil and De vries in 1967 (Cunge et al. 1980), several one dimensional movable bed models have been developed. Holly (1986), Dawdy and Vannoni (1986), Cunge et al. (1980) and Palaniappan (1991) presented excellent reviews of numerical simulation of alluvial hydraulics. Lu and Shen (1986) tested several numerical aggradation and degradation models by comparing the computed results with the laboratory data obtained by Suryanarayana (1969). Most of the standard one dimensional unsteady sediment transport models can be classified into either uncoupled models, wherein the water flow equations and sediment continuity equation are uncoupled during a given time step or coupled models, wherein all the governing equations are solved simultaneously. They can also be classified based on whether full Saint Venant equations for shallow water flow are considered (Unsteady flow models), or the water flow is assumed steady during the computations of bed level variation (Quasi steady flow models).

Quasi-steady flow uncoupled models: HEC-6, developed by US Army corps

of Engineers (Thomas and Prashun 1977, Thomas 1982, Copeland 1986), Flvial-3 (Chang and Hill 1976), KUWASER (Simons et al. 1979), IALLUVIAL (Karim and Kennedy 1982, Holy et al. 1984, Karim 1985, Holly and Karim 1986) and a recent model developed by Palaniappan (1991) are good examples of quasi-steady uncoupled models. Witkowska (1971), Bettess and White (1979, 1981), and Young (1984) also developed similar models. In all these models, one- dimensional steady flow equations are used to compute the gradually varied flow profile in a channel. The sediment mass balance equation is then used to determine the bed level change during the specified computational time step. This is followed by recomputation of the water surface profile corresponding to new bed levels, and the procedure is repeated. Models belonging to this class differ from one another in terms of (1) the equation for the sediment transport capacity, (2) the equation used for computing the friction slope, (3) the finite difference approximation of the sediment mass balance equation, (4) the level of detailing the cross-sectional shape and, (4) the armouring procedure. For example, Witkowska (1971) ignores the armouring procedure completly while Holly and Karim (1986) include, probably the most sophisticated procedure for simulating the armouring phenomenon. Recently, Mohapatra and Bhallamudi (1994) used a quasi-steady flow model for studying the be level variation in channel transitions.

Quasi-steady Flow Coupled Models: CHAR-3 (Cunge and Simons (1975) and CHAR-2 (1978) belong to this class of mobile bed modeling. The steady flow momentum equation for water and the unsteady mass balance equation for the sediment transport equation are solved using the Preismann implicit scheme. The resulting nonlinear algebraic equations are solved simultaneously to achieve coupling between the water flow and the sediment movement. CHAR-2 and CHAR-3 are based on basic modeling philosophy proposed by Cunge and Perdreau (1973).

Unsteady Flow Uncoupled Models: Quasi-steady flow models can not be

applied strictaly while studying flood related changes. Complete Saint Venant equations for water flow should be considered in such cases. FLUVIAL programs developed by Chang (Chang 1982,1984,1985),the model developed by Bennet Nordin at USGS (1977), the models developed at the Colorado State University (Chen 1973, Chen et al. 1975, Chen and Simons 1980) and CHAR-1 (Cunge et al. 1980) are examples of unsteady, uncoupled models. The water flow and the sediment transport computations lag by one computational time step in all these models.

Unsteady Flow Coupled Models: Vreugdenhil (1982) and Lyn (1987) studied the stability characteristics of one dimensional movable bed modeling. Lyn (1987) used perturbation techniques to identify multiple time scales in the governing equations and suggested that complete coupling between the full unsteady water flow equations and the sediment continuity equation is desirable in cases where the conditions are rapidly changing at the boundaries. The models developed by Chen and Simons (1975), Chollet (1977) and Krishnappan (1981,1985) belong to the class of one dimensional, unsteady, coupled models. Park and Jain (1986) achieved a partial coupling between sediment flow and water flow by iterating the solution whenever the spatial gradient of change in bed level became too large in order to avoid numerical instabilities. Later, Park and Jain (1987) introduced a simple bed layer model to simulate the armouring process. Lai and Chang (1987) developed an unsteady model using characteristic method with implicit and reach back schemes. Havno et al. (1985) modified the model MIKE-11 developed at the Danish Hydraulic institute (DHI 1983) to include the erosion procedures. In this model, the St.Venant equations are solved using a fully centered implicit difference scheme. Rahuel (1988) and Rahuel et al. (1989) developed a fully coupled implicit finite difference model which includes a procedure for routing the graded mixtures. This model requires the solution of (2k+1) partial differential equations, where k is the total number of sediment size classes. Later, Holly and Rahuel (1990) extended this model to treat

suspended and bed load transports separately. It should be noted that this model was deveĺoped for a single channel only. It requires enormous data preparation and computational power for application to even single channels. Extensions of this model to channel networks is a very difficult task. Correia et al. (1992) also developed a fully coupled unsteady mobile boundary flow model for application to a single channel. In this model, the governing equations are formulated in such a way that water flow equations and sediment transport equation are explicitly coupled. The equations are solved using the Priessman scheme. However, a new method is introduced for solving the simultaneous linear equations which result from the discritization. It should be noted here that most of the unsteady, coupled models use implicit schemes to solve the governing equations. Although, large computational time steps can be used with these schemes, they involve solution of a system of equation by matrix inversion during each computational time step. This results in high computational costs, especially when extended to the case of channel systems. Complete unsteady flow modeling is usually required only when studying the flood related changes. Computations in these cases may have to be performed for time periods of at the most for two to three months. Therefore, use of explicit methods may not be a bad choice. Bhallamudi and Chaudhry (1991) presented one such completely coupled unsteady flow model using the second order accurate Maccormack scheme. This model has been used to study (1) the aggradation due to sediment overloading, (2) the degradation due to base level lowering and, (3) the knick-point migration in laboratory channels.

Although the present trend is to use 'coupled models' for reverine studies, some researchers feel that the concerns in the literature about the use of coupled models may have been overstated. Recently, Cui et al. (1996) compared the numerical results obtained using coupled and quasi-steady uncoupled models for cases with Froude number very close to unity and also for cases in which the upstream water

discharge, sediment feed rate and downstream water elevation varied strongly. There was a very good agreement between the numerical results obtained using the two models, although uncoupled models are inherently more unstable than the coupled models. Performance of uncoupled models appears to be satisfactory for most cases of riverine problems. It should be noted that quasi-steady flow uncoupled models require significantly less computational power for their application. Therefore, they are ideally suited for application to large river network problems and for simulating long term effects. It is also easy to include sophisticated relations for sediment transport capacity, armouing process and friction factor in these models. Therefore, a quasi-steady flow uncoupled approach is followed in the present study.

Channel Network Models: There has not been much research in the area of sediment routing in channel networks. In fact, studies on even a basic topic such as the gradually varied flow (GVF) computation in channel networks are few and recent. Shulte and Chaudhry (1986) presented a numerical algorithm for computing water-surface profiles in steady-state, gradually varied flows in open channel networks. In this procedure, channel in the network is discritized into finite difference nodes and the energy equation is written for each pair of two consecutive finite difference nodes. These equations along with the compatibility equations for the channel junctions are simultaneously solved using the Newton-Raphson technique. This method can be used for both tree type and looped networks. However, the structure of network is not exploited in this algorithm to achieve computational efficiency. Also, the convergence of the Newton-Raphson technique is not guaranteed.

Many researchers suggest the use of false transient approach to obtain steady state solutions in the networks (Holly et al. 1985). Research interest in the area of unsteady flow modeling in channel networks has been revived in recent years. The commercial model FLDWAV (Fread 1985) is applicable for river networks. However,

it does not exploit the special structure of tree type networks and therefore, is computationally very intensive. Application of implicit methods for unsteady flow in channel networks becomes very difficult because the double sweep algorithm can not be applied in a straight forward manner. Akan and Yen (1981) applied an iterative technique in routing floods through tree type channel networks. However, they used the governing equations for diffusion routing and therefore, the backwater effect of a junction can not be considered. If the down stream backwater effect is significant, a large number of iterations may be necessary to achieve the desired accuracy. Tucchi (1978) and Barkau et al. (1989) applied the skyline algorithm for computing unsteady flow in channel networks. The skyline algorithm requires a large number of iterations during the reduction pass. In recent years, there has been an emphasis on solving the entire network as a unit system rather than as a combination of independent systems. Choi and Molinas (1973) introduced one such direct solution algorithm based on the double sweep method. This method is applicable to dendritic channel networks. The model proposed by Choi and Mohinas (1993) requires operation on a 2N $\times$ 4 matrix as compared to the operation on a 2N $\times$ 2N matrix (N = number of cross sections used) in the direct application of the Preismann scheme. In this model, the channel network problem is converted into a single channel problem by using separate recursive equations for each of the components of the network i.e, (1) interior channel cross sections, (2) converging channel junction and, (3) diverging channel junction. Nguyen and Kawano (1995) proposed an alternative algorithm in which all the channel sections and junctions can be uniformly treated. The model proposed by Nguyen and Kawano (1995) is applicable to a broader range of dendritic type of networks. Recently, Tiwari (1996) slightly modified the algorithm presented by Nguyen and Kawano (1995) to take into consideration Log Pearson Type-III hydrograph as the inflow boundary and a barrage as the downstream boundary conditions. The algorithm was successfully

applied to a channel network as large as the Ganga system from Hardwar to Farakka with all its tributaries. The algorithm was applied to simulate flows for one month during the flood season. It should be noted here that algorithms discussed so far are applicable for simulating unsteady flow in dendritic systems only. On the other hand, Kutija (1995) applied graph theoretic concepts and computer algebraic procedures to develop an algorithm for computing unsteady flow in combined dendritc and looped channel networks. The complex algorithmic structure proposed by Kutija (1995) is required only for application to large urban drainage systems where channels are multiply connected. Most of the natural river systems are tree type except in the deltaic region where a looped structure may exist. Therefore, algorithms which exploit this tree structure are most useful in the present context. It should also be noted here that all the algorithms for unsteady flow in networks discussed so far are based on the implicit finite-difference method. On the other hand, Misra et. al. (1989,1992) developed an explicit numerical model, TRANS-NET for study of unsteady flow in irrigation canal networks. Space line interpolation explicit characteristic scheme on rectangular grids is used for solving the Saint Venant equations. This model can be also used for flood routing in river networks.

All the network models described so far are applicable only for flood routing. Network models for sediment routing are very few. A commercial model called UUWSR was developed at the University of Colorado (Chen and Simons, 1980) for routing floods and sediment through the channel networks. This model is based on the complete unsteady flow equations but follows a decoupled approach. Holly et. al. (1985) and Yang (1986) also developed a network model for flood and sediment routing in multiply connected channels. Their model is based on the quasi-steady assumption for the water flow and uses the uncoupled approach. The quasi-steady flow conditions, however, are obtained using the unsteady flow equations in a false transient approach. It should be noted here that use of a steady flow network solver

in a quasi-steady flow uncoupled sediment routing model would result in significant saving of computational time. The steady flow network solver, recently developed by Naidu et. al. (1997) is ideally suited for this purpose. Recognising that the tree type networks are more common that the looped networks, Naidu et. al. (1997) developed a very efficient algorithm for computing GVF profiles in tree type channel networks. This algorithm is based on some basic graph theoretic concepts and the shooting method for the boundary value problems. It was shown that the proposed method could result in significant savings in computational costs, some time as high as 200 times when compared with the steady-flow solver developed by Shulte and Chaudhry (1986). It should be noted here that the algorithm developed by Naidu et. al. (1997) is designed for computing GVF profiles in diverging channel networks where water from a main channel gets distributed to branch channels. Therefore, a sediment routing model based on the network solver developed by Naidu et. al. (1997) would be ideally suited for evaluating aggradation and degradation in (1) channels in deltaic region and, (2) unlined irrigation canal systems. This method is not efficient for converging channel systems where the flows from many tributaries join together to form the main river. However, computation of GVF profiles in converging channel systems is not complicated since it is possible to determine the flow in each channel as the solution to an initial value problem.

1.3 Objectives of the Present Study

The literature review presented in the previous section shows that the majority of the aggradation and degradation models are applicable for single channels. Only few models have been developed to study the bed level variations in multiple channel systems. Objective of the present study is to develop quasi-steady flow, uncoupled models for aggradation and degradation in tree-type channel networks. Two separate

numerical models are developed for the cases of converging and diverging channel systems, respectively. Very efficient numerical procedures are adopted while developing these numerical models. Also, general cross-sectional shapes and a sophisticated predictor for the sediment transport capacity are considered. This work is part a on going effort at I. I. T. Kanpur (supported by the Ministry of Water Resources, India) to develop a comprehensive mathematical modeling system for flood and sediment routing in channel networks

The following chapters present the theoretical considerations and, numerical procedures . Applicability of the model is demonstrated in chapter 4. The thesis concludes with a summary and recommendations for further work.

Chapter 2

Theoretical Considerations

Simulation of aggradation and degradation in open-channels with movable beds involves the numerical solution of governing differential equations for water and sediment flow. Inspite of extensive research to understand the relationship between water flow and sediment transport (Shen 1971, Garde and Ranga Raju 1985), a satisfactory analysis from the point of view of basic mechanics is still out of reach. The present understanding of sediment transport is highly empirical in nature. In addition, numerical solution of three-dimensional equations of motion for water and sediment to simulate the actual movement of sediment is quit complicated, and is not necessary from the engineering point of view. Therefore, like most of the existing models, aggradation and degradation model presented in this study is based on the following considerations.

1. We are only interested in computing the bed level changes and, not in computing the velocity of sediment particles.

2. Sediment transport capacity can be satisfactorily quantified by a functional relation with average flow parameters.

With the above assumptions, a sediment continuity equation and an equation for the sediment transport capacity as a function of flow completely describe the sediment flow. Because sediment discharge is related to average flow parameters, it suffices to represent the water flow by cross sectional averaged quantities.

As already mentioned in chapter - 1, it appears that quasi-steady flow uncoupled models do give satisfactory results in most of the cases of practical interest (Cui et al. 1996). The numerical model presented in this study belongs to that category. Quasi-steady flow models are based on the fact that, in general, the free surface wave celeraties are far greater than the celerity of bottom perturbation (Cunge et al. 1980). Therefore, it is possible to study bed perturbations while neglecting the time derivative terms in the water flow equations. This means that the water flow may be represented by a single steady state equation such as the energy equation.

The other major assumption used in the present model is that the sediment is of uniform size. The size characteristic of the sediment is represented by the median diameter,d_{50}. This assumption is not valid in cases where the sediment is well graded. During degradation, finer particles are transported in preference to larger particle. A stage may be reached when the bed has only sufficiently large particles which can not be moved by the flow. These large particles on the bed surface provide a protective layer for the underlying finer particles which would otherwise be transported. This phenomenon is known as armouring. Since the proposed model considers only a uniform sediment size, it can not be used where degradation is controlled by armouring. This aspect is being looked into a parallel study at IIT Kanpur. It should also be noted here that, like most of the currently available models, only bed load transport is considered and the effect of suspended sediment load is assumed to be negligible.

In height of the above discussion, the essential feature of the aggradation and degradation models is the solution of four coupled relations :

1. Energy equation for water flow

2. A friction-slope equation

3. Equation for sediment discharge and ,

4. Equation of continuity for sediment.

The following sections present details of these formulation aspects of the model.

2.1 Equation for Water Flow

Equation for water flow is based on the following assumptions :

1. the pressure distribution in a cross-section is hydrostatic,

2. the channel bottom slope is small,

3. the energy correction factor, $\alpha = 1.0$,

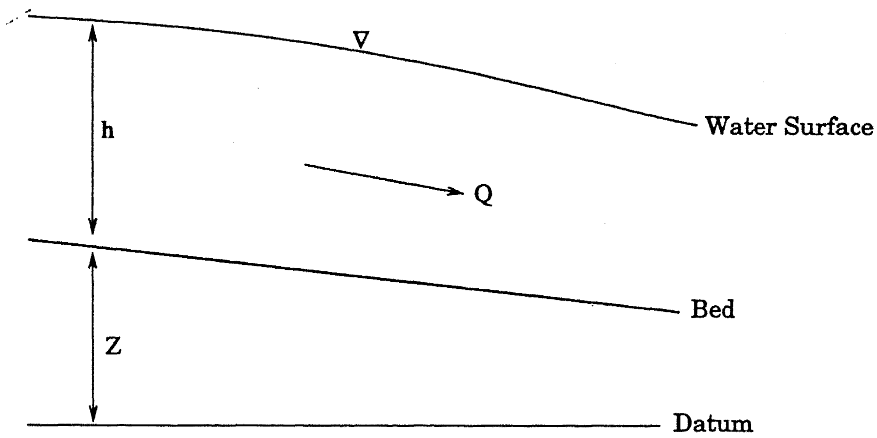
4. all the channels in the system are prismatic to start with, and remain more or less prismatic even after aggradation or degradation and,

5. the flow is steady.

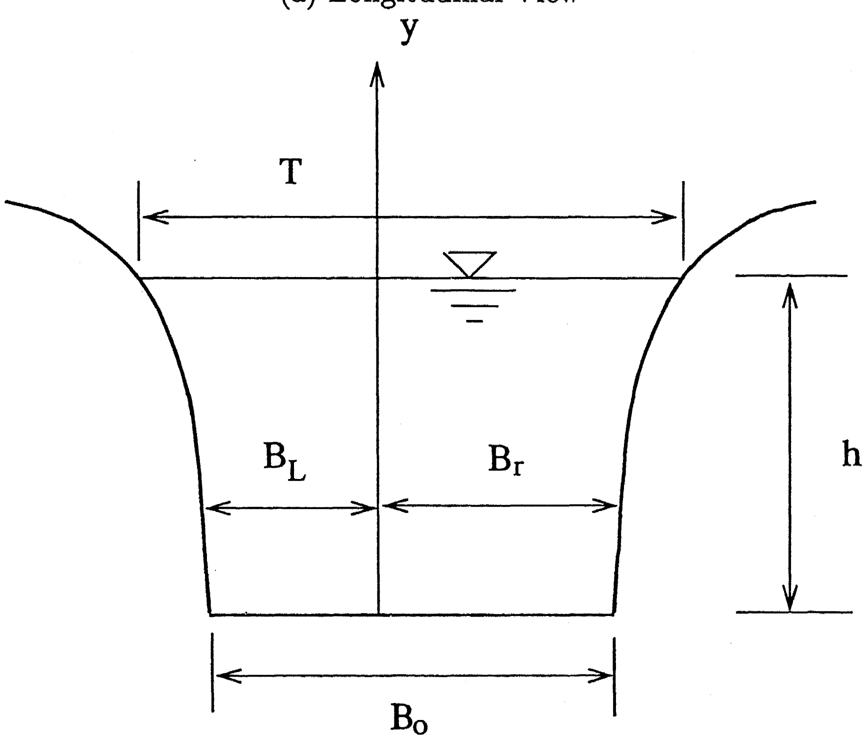
The steady state energy equation for a channel reach, as shown schematically in Fig. 2.1, can be expressed as (Chaudhry 1993, Subramanya 1997).

$$\frac{dh}{dx} = \frac{\frac{dZ}{dx} - S_f}{1 - \frac{Q^2T}{gA^3}} \tag{2.1}$$

Where h = flow depth (m), Z = bed elevation with respect to a specified datum (m), Q = discharge (m^3/sec), T = water surface width (m), A = cross-sectional area (m^2), g = acceleration due to gravity (m/s^2), S_f =the friction slope and, x =



(a) Longitudinal View



(b) Cross - Section

Figure 2.1: Schematic view of the channel

distance along the channel (m). A general irregular cross- section as shown in Fig 2.1b is considered in the present study. The channel width, B at any height y above the bed is assumed to be given as

$$B = B_0 + C_1 y^{C_2} + C_3 y^{C_4} \tag{2.2}$$

where, B_0 = the channel bottom width, C_1, C_2, C_3 and C_4 are coefficients which are determined by the channel shape. The initial values of C_1, C_2, C_3 and C_4 any for channel section at any distance may be obtained using the field survey data. Curve fitting methods may be used for this purpose. These values are continuously updated in the numerical model as the aggradation or the degradation of the section occurs. The top width, T is now given by

$$T = B_0 + C_1 h^{c_2} + C_3 h^{C_4} \tag{2.3}$$

The cross-sectional area A can be expressed as

$$A = B_0 h + \frac{C_1}{1+C_2} h^{1+C_2} + \frac{C_3}{1+C_4} h^{1+C_4} \tag{2.4}$$

The wetted perimeter P can be expressed as

$$P = B_0 + \int_0^h \left[\sqrt{(C_1 C_2)^2 y^{(2C_2-2)} + 1} + \sqrt{(C_3 C_4)^2 y^{(2C_3-2)} + 1} \right] dy \tag{2.5}$$

It should be noted here that C_1 and C_3 are taken as zero in the case of rectangular channels. C_2 and C_4 are taken equal to one in the case of unsymmetrical trapezoidal channels. The channel shape obtained using Eq. (2.2) is a cross between a

trapezoidal and an exponential channel. Eq. (2.2) and thus, the present numerical model cannot be applied for the case of channels with flood plains. An attempt is made in the present study to include the compound channel shapes also. However, the work remained incomplete due to lack of time.

2.2 Sediment Continuity Equation

The aggradation or the degradation in a channel reach is calculated by applying the sediment continuity equation between the two bounding sections. The sediment continuity equation is derived by applying the principle of conservation of mass to a differential control volume surrounding the channel bed. It should be noted here that only the bed load transport is considered in this study. The sediment continuity equation is expressed as (Raudkivi 1990)

$$(1-p)\frac{\partial A_s}{\partial t} + \frac{\partial Q_s}{\partial x} = 0 \tag{2.6}$$

where A_s = the cross-sectional area surrounding the channel bed through which the bed load transport takes place, Q_s = total bed load discharge through the cross-section, P = porosity and t = time. For a rectangular channel, Eq. (2.6) reduces to

$$(1-p)\frac{\partial Z}{\partial t} + \frac{\partial q_s}{\partial x} = 0 \tag{2.7}$$

where, q_s = bed load transport per unit width.

2.3 Sediment-Discharge and Friction-slope Predictors

There are no analytical equations which relate the friction-slope, S_f and the sediment discharge Q_s to flow and the sediment characteristics. Therefore, empirical equations are generally used. Several equations are available for this purpose. A good review of these equations can be found in the literature (Graf 1971, Vanoni 1975). Jansen et al. (1979) point out that many of the sediment discharge formulae can be expressed in a functional form as

$$X = f(Y) \tag{2.8}$$

Where, X is a transport parameter which depends on the sediment discharge and the grain properties. Y is a flow parameter which depends on the flow properties and the representative grain diameter. The numerical model presented in this study is structured in such a way that any sediment discharge relationship can be easily considered. At the present stage of development, three alternative procedures are included in the numerical code, which are described as follows.

2.3.1 Power Law Model

In this model, the sediment discharge is estimated by an empirical power function of the flow velocity. The unit sediment discharge, q_s is given by

$$q_s = a\left(\frac{Q}{A}\right)^b \tag{2.9}$$

where a and b are empirical constants whose values depend upon the sediment properties. It should be noted here that Eq. (2.9) has been used earlier by many researchers (Soni et al. 1980, Zhang and Kahawita 1987, Bhallamudi and Chaudhry

1991). In the power law model, the friction-slope, S_f is estimated using the Manning's equation.

$$S_f = \frac{Q^2\, n^2\, P^{\frac{4}{3}}}{A^{\frac{10}{3}}} \tag{2.10}$$

where, n = Manning's roughness coefficient.

2.3.2 Meyer–Peter–Müller equation

Meyer-Peter-Müller formula is very popular in Europe. It was developed based on experiments with coarse sand and gravel, of different relative densities. Most of the data upon which the formula is based were obtained in flow conditions with little or no suspended load. This equation can be expressed as (Chien 1954)

$$\Phi = 8\left[\frac{\tau'}{(\gamma_s - \gamma_w)\, d_{50}} - 0.047\right]^{1.5} \tag{2.11}$$

$$\Phi = \left(\frac{q_b}{\gamma_s}\right)\left[\frac{\gamma_w}{(\gamma_s - \gamma_w)\, d_{50}}\left(\frac{1}{g\, {d_{50}}^3}\right)\right]^{0.5} \tag{2.12}$$

where, τ' = particle shear stress, d_{50} = median diameter of sediment particles, γ_w = unit wt. of water, γ_s = unit wt. of sediment particles and q_b = sediment transport rate in weight per unit time per unit width. It should noted that Eq. (2.11) and Eq. (2.12) are in English units. As a first approximation, the particle shear stress, τ' can be taken as bed shear stress τ_b which is given as

$$\tau_b = \rho g \left(\frac{A}{P}\right) S_f \tag{2.13}$$

where, ρ = density of water. The friction -slope, S_f in Eq. (2.13) may be estimated using Eq. (2.10)

2.3.3 Iowa Model

Karim and Kennedy (1981) of the University of Iowa, developed a Total Load Transport Model(TLTM) for the sediment discharge and friction factor predictors. TLTM is derived by using non linear multiple regression analysis to a large body of laboratory and field data. This formulation takes in to account the well known fact that the friction-slope in alluvial channels is heavily dependent on the sediment discharge. The following sediment discharge and friction-slope predictors from TLTM are adapted as one of the alternatives in the present study.

Sediment discharge predictor:

$$\begin{aligned}\log\left[\frac{q_s}{\sqrt{g\,(s-1)\,{d_{50}}^3}}\right] &= -2.2786 + 2.9719\,\log V_1 \\ &\quad + 1.06\,\log V_1\,\log V_6 + 0.2989\,\log V_2\,\log V_6 \qquad (2.14)\end{aligned}$$

Friction slope predictor:

$$\begin{aligned}\log\left[\frac{Q}{A\sqrt{g\,(s-1)\,d_{50}}}\right] &= 0.9045 + 0.1665\,\log V_7 \\ &\quad + 0.0831\,\log V_4\,\log V_5\,\log V_7 + 0.2166\,\log V_4\,\log V_5 \\ &\quad - 0.0411\,\log V_2\,\log V_3\,\log V_4 \qquad (2.15)\end{aligned}$$

where,

$$V_1 = \frac{Q}{A\sqrt{g\,(s-1)\,d_{50}}} \qquad (2.16)$$

$$V_2 = \frac{h}{d_{50}} \qquad (2.17)$$

$$V_3 = S_f * 10^3 \qquad (2.18)$$

$$V_4 = \frac{\sqrt{g\,h\,S_f}}{\omega} \qquad (2.19)$$

$$V_5 = \frac{\omega\, d_{50}}{\nu} \tag{2.20}$$

$$V_6 = \frac{\sqrt{g\, h\, S_f} - u_{*c}}{\sqrt{g\,(s-1)\, d_{50}}} \tag{2.21}$$

$$V_7 = \frac{q_s}{\sqrt{g\,(s-1)\, {d_{50}}^3}} \tag{2.22}$$

In Eqs. (2.14)- (2.22), ω =fall velocity of sediment particles, ν = kinematic viscosity of water, s = specific gravity of sediment particles and u_{*c} = critical shear velocity obtained from the Sheilds diagram. Following explicit equation for u_{*c} (Raudkivi 1990) is used in the present study

$$d_* = 2.15\, R_{e*} \qquad R_{e*} < 1 \tag{2.23}$$

$$d_* = 2.5\,(R_{e*})^{0.8} \qquad 1 < R_{e*} < 10 \tag{2.24}$$

$$d_* = 3.8\,(R_{e*})^{0.625} \qquad R_{e*} > 10 \tag{2.25}$$

In Eqs. (2.23-2.25), d_* = dimensionless particle size = $[\,(\gamma_s - \gamma)\, g\, {d_{50}}^3/\nu^2\,]^{\frac{1}{3}}$, and R_{e*} =particle Reynolds number = $\sqrt{g\,\frac{A}{P}\, S_f} \,*\, \frac{d_{50}}{\nu}$

In all the three prior discussed models, equations are available only for determining the unit sediment discharge. Therefore, these models can be readily applied in case of rectangular channels and wide channels. It should noted here that no predictive equations are available for estimating total sediment discharge(Q_s) in narrow irregular channels. Q_s in these channels is assumed to be given by the following equation.

$$Q_s = T\, q_s\, C_5 \tag{2.26}$$

where, C_5 is an empirical coefficient which depends upon the shape of the cross-section. C_5 is taken equal to one in all the simulations reported herein.

Chapter 3

Computational Procedure

Several numerical techniques are adopted to solve the governing equations of quasi-steady flow, uncoupled model for application to tree-type alluvial channel networks. The details of these procedures are discussed in this chapter. The overall algorithmic structure of the model is also presented.

3.1 Determination of B_0, C_1, C_2, C_3 and C_4

The coefficients B_0, C_1, C_2, C_3 and C_4 in Eq. (2.3) are determined from the known values of widths B(y) at some discrete points along the x-axis. Linear regression analysis is adopted for this purpose. Width, $B_l(y)$ is divided into two parts B_r and B_l where, B_r = the distance between the right bank and the y- axis while B_l = the distance between the left bank and the y- axis. The curve for the left bank can now be expressed in a functional form as

$$B_l(y) - B_l(0) = C_1 y^{C_2} \tag{3.1}$$

Equation (3.1) may be transformed to a linear relationship as follows

$$x' = C_1' + C_2 y' \tag{3.2}$$

where, $x' = \ln[B_l(y) - B_l(0)]$, $y' = \ln(y)$, $C_1' = \ln(C_1)$. C_1' and C_2 can be determined from the known values of x' at some discrete points along the x- axis using the least squares linear regression (Press et al. 1986). The equations are

$$C_2 = \frac{M\overline{y'}\,\overline{x'} - \sum_{i=1}^{M} y_i' x_i'}{M(\overline{y'})^2 - \sum_{i=1}^{M} y_i'^2} \tag{3.3}$$

and

$$C_1' = \overline{x'} - C_2\overline{y'} \tag{3.4}$$

where M = the total number of points at which $B_l(y)$ is known, and the overbar indicates the mean values. Value of C_1 can be easily determined from C_1'. The same procedure is adopted for determining C_3 and C_4 from the known values of $B_r(y)$. It should be noted that the values of B_0, C_1, C_2, C_3 and C_4 are updated at the end of each computational time step because the aggradation or degradation at the cross-section may alter the values of $B_l(y)$ and $B_r(y)$.

3.2 Computation of Wetted Perimeter, P

The cross -sectional area, A corresponding to a known value of flow depth, h can be easily computed using Eq.(2.4). However the computation of the wetted perimeter, P using Eq. (2.5) is not trivial. Five point Gaussian quadrature integration as described below is used for this purpose. Equation (2.5) may be recast as

$$P = B_0 + \int_0^h f(y)\,dy \tag{3.5}$$

where

$$f(y) = \sqrt{1 + (C_1C_2)^2\, y^{2C_2-2}} + \sqrt{1 + (C_3C_4)^2\, y^{2C_4-2}} \tag{3.6}$$

k	1	2	3	4	5
W_k	0.236926885	0.478628670	0.568888889	0.478628670	0.236926885
Y_k	-0.906179846	-0.538469310	0.0	0.538469310	0.906179846

Table 3.1: W_k and Y_k for the Gaussian quadrature

Eq. (3.5) can noe be approximated by the five point Gaussian quadrature as (Press et al. 1986)

$$P = B_0 + \frac{h}{2}\sum_{k=1}^{5} W_k f(y_k') \tag{3.7}$$

where, W_k = value of the weight corresponding to the k'th point, and y_k' = normalized value of y at the k'th point.

$$y_i' = \frac{h}{2}(1 + Y_k) \tag{3.8}$$

W_k and Y_k values for $k = 1$ to 5 are given in the table 3.1

3.3 Computation of Sediment–discharge and Friction–slope

Computation of sediment–discharge, q_s and friction-slope , S_f for the known flow and sediment characteristics is trivial in the case of Power law and the Meyer-Peter-müller models. However, this is not so in the case of TLTM proposed by Karim and Kennedy (1982). Equations (2.14) and (2.15) are solved using an inefficient first-order iteration technique in the IALLUVUIAL model devopled by Karim and Kennedy (1982). In this study, the Newton-Raphson technique is used to solve for the q_s and S_f in the TLTM model.

A single equation for the unknown S_f is first derived by eliminating q_s from Eqs. (2.14) and (2.15). Denoting

$$\begin{aligned}
\sqrt{g\,(s-1)\,d_{50}} &= a_1 && (3.9)\\
\sqrt{g\,h} &= a_2 && (3.10)\\
\log(\frac{Q}{A\,a_1}) &= a_3 && (3.11)\\
\log(\frac{h}{d_{50}}) &= a_4 && (3.12)\\
\log(\frac{q_s}{a_1\,d_{50}}) &= x_1 && (3.13)\\
a_2\sqrt{S_f} &= x_2 && (3.14)
\end{aligned}$$

Eq. (2.14) can be written as

$$\begin{aligned}
x_1 &= [-2.2786 + 2.9719\,a_3 - (1.06\,a_3 + 0.2989\,a_4)\log(a_1)]\\
&\quad + (1.06\,a_3 + 0.2989\,a_4)\log(x_2 - u_{*c}) && (3.15)
\end{aligned}$$

Denoting

$$-2.2786 + 2.9719\,a_3 - (1.06\,a_3 + 0.2989\,a_4)\log(a_1) = a_5 \qquad (3.16)$$

and

$$1.06\,a_3 + 0.2989\,a_4 = a_6 \qquad (3.17)$$

Eq. (3.15) can be written as

$$x_1 = a_5 + a_6\log(x_2 - u_{*c}) \qquad (3.18)$$

Denoting

$$\log\left(\frac{\omega d_{50}}{\nu}\right) = a_7 \qquad (3.19)$$

$$\log\left(\frac{1000}{a_2 * a_2}\right) = a_8 \tag{3.20}$$

$$0.1665 - 0.0831\, a_7 \log(\omega) = a_9 \tag{3.21}$$

$$0.2166\, a_7 - 0.0411\, a_8 + 0.0822 \log(\omega) = a_{10} \tag{3.22}$$

$$0.0831\, a_7 = a_{11} \tag{3.23}$$

$$-0.0822\, a_4 = a_{12} \tag{3.24}$$

$$0.9045 - 0.2166\, a_7 \log(\omega) + 0.0411\, a_4\, a_8 \log(\omega) - a_3 = a_{13} \tag{3.25}$$

and substituting Eqs. (3.9)-(3.14) and (3.19)-(3.25) in Eq. (2.15) one can obtain

$$a_9\, x_1 + a_{10} \log(x_2) + a_{11}\, x_1 \log(x_2) + a_{12}\{\log(x_2)\}^2 + a_{13} = 0 \tag{3.26}$$

Denoting

$$a_9\, a_5 + a_{13} = b_1 \tag{3.27}$$

$$a_9\, a_6 = b_2 \tag{3.28}$$

$$a_{10} + a_5\, a_{11} = b_3 \tag{3.29}$$

$$a_{11}\, a_6 = b_4 \tag{3.30}$$

and substituting Eq. (3.18) in Eq. (3.26) one can obtain

$$\begin{aligned} f(x_2) &= b_1 + b_2 \log(x_2 - u_{*c}) + b_3 \log(x_2) \\ &\quad + b_4 \log(x_2) \log(x_2 - u_{*c}) + a_{12}(\log(x_2))^2 = 0 \end{aligned} \tag{3.31}$$

Coefficients b_1, b_2, b_3, b_4 and a_{12} in Eq. (3.31) are functions of known flow and sediment characteristics, h, Q, d_{50}, ν and ω. The critical shear stress, u_{*c} is also determined explicitly using the median diameter of the sediment particles, d_{50}. The only unknown in Eq. (3.31) is x_2, and it can be solved for using the Newton-Raphson technique. In this method, a value is first assumed for the x_2, and is then improved iteratively using the following equation.

$$x_2^{new} = x_2^{old} - \frac{f(x_2)}{f'(x_2)} \tag{3.32}$$

where,

$$\begin{aligned} f'(x_2) &= \frac{df(x_2)}{dx_2} \\ &= \frac{b_2}{x_2 - u_{*c}} + \frac{b_3}{x_2} + \frac{b_4 \log(x_2 - u_{*c})}{x_2} \\ &+ \frac{b_4 \log x_2}{x_2 - u_{*c}} + 2\,a_{12}\frac{\log(x_2)}{x_2} \end{aligned} \tag{3.33}$$

A very small value for the error tolerance is used while solving the Eq. (3.31)by the Newton-Raphson technique. The friction-slope, S_f is computed using Eq. (3.14) once the value of x_2 is determined. Subsequently, q_s can be determined using Eqs. (3.18) and (3.13). It should be noted here that this minor modification to the way the TLTM is applied results in significant savings in the total computational time because S_f needs to be computed at least four times at each section during each computational time period.

3.4 Solution Of Gradually Varied Flow Equation

Equation (2.1) representing the steady gradually varied flow in open-channels is used to determine the flow depth at any section 'j' in channel 'i' once the flow conditions at the downstream section 'j+1' in the same channel are known. Fourth-order Runge-Kutta method (Chaudhry 1993, Subramanya 1997) is used to solve Eq. (2.1) numerically.

$$h_{i,j} = h_{i,j+1} - \frac{\Delta x}{6}(SL_1 + 2SL_2 + 2SL_3 + SL_4) \tag{3.34}$$

where, the slopes SL_1, SL_2, SL_3 and SL_4 are determined using the following procedure.

1. For the known discharge, Q_i and the flow depth $h_{i,j+1}$ at section $i+1$, compute the flow area, A, top width, T, and the wetted perimeter, P using the geometric properties of the cross–section (i,j+1). Eq. (2.3), (2.4) and the procedure given in section 3.2 are used for this purpose, respectively.

2. S_f at section $(i, j+1)$ is computed using either the Manning's equation or the procedure described in section 3.3 for the TLTM model.

3. Above values of A, T and S_f are used in Eq. (2.1) to compute the $SL_1 = \frac{dh}{dx}$ value.

4. A predicted flow depth, h' is determined as

$$h' = h_{i,j+1} - \frac{\Delta x}{2}(SL_1) \tag{3.35}$$

where, $\Delta x =$ the distance between the section (i,j) and (i,j+1).

5. Using the h' value as determined from Eq. (3.35) and the geometric properties of the section midway between (i,j) and (i,j+1), steps 1-3 are repeated to determine the value of SL_2.

6. A new predicted flow depth, h'' is determined as

$$h'' = h_{i,j+1} - \frac{\Delta x}{2}(SL_2) \tag{3.36}$$

7. Steps (1)-(3) are repeated using h'' and the section properties at $(i, J + \frac{1}{2})$ to determine the value of SL_3.

8. A predicted flow depth, h''' is determined as

$$h''' = h_{i,j+1} - \Delta x\,(SL_3) \qquad (3.37)$$

9. steps 1-3 are repeated using h''' and the section properties at (i,j) to determine the value of SL_4

Runge-Kutta method is applied in a marching fashion from the node(i,n_i) to the node (i,1) on the channel i. To start the computations, the discharge in the channel, Q_i and the flow depth at the last node of the channel, h_{i,n_i} should be specified. h_{i,n_i} is known either from the specified downstream boundary condition for a pendant channel or from the previous computations.

3.5 Computation Of GVF Profiles in Channel Networks

A large part of the computational expenditure in the application of the aggradation or degradation model to alluvial channels is incurred by the water surface profile computations. This is especially true in the case of large channel networks. Therefore, the major emphasis in the present study is to perform these computations as efficiently as possible. Application of the fourth-order Runge–kutta method as described in section 3.4 is straight forward in the case of single channels. However, this is not so in the case of channel networks. In this study two separate computer codes are developed for (1) the converging tree-type channel networks and, (2) the diverging tree-type channel networks, respectively, by exploiting the special structures inherent in these problems.

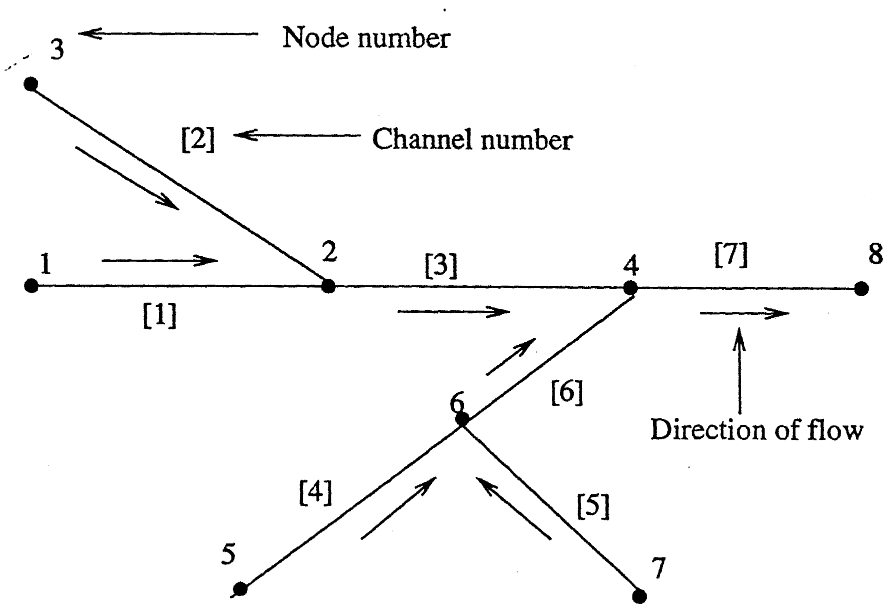


Figure 3.1: Typical converging tree-type network

3.5.1 Converging Tree-type Channel Networks

In a typical converging tree-type channel network (Fig. 3.1), flows from several tributaries join together to form the flow in the main channel. This is typical of river systems in non-deltaic regions. For the sake of simplicity, it is assumed that (1) discharge from the system leaves through one single channel at the downstream end, (2) there are no hydraulic structures within the system, (3) only two channels feed into single channel at a junction and, (4) the flow is subcritical throughout the system.

Assumptions (1) and (3) apply to most of the natural channel systems while the assumption (2) can be easily relaxed to include intermediate control structures. The occurrance of the supercritical flow in channels is rare and requires special treatment. Analysis of such a system is beyond the scope of the present study. The problem of GVF computation in converging tree-type channel systems can be defined as

"given the discharges in all the root channels(Q_1, Q_2, Q_4 and Q_5 in Fig. 3.1) and the flow depth at the pendant node(h_8 in Fig. 3.1),compute the water surface profile in the system".

The above problem can be solved by the successive application of the procedure for single channel, termed as the Initial Value Problem(IVP) hereafter, and by applying the compatibility conditions at a junction. The compatibility conditions at a junction are given by

$$Q_i + Q_j = Q_k \tag{3.38}$$

and

$$h_{i,n_i} + Z_{i,n_i} = h_{j,n_j} + Z_{j,n_j} = h_{k,1} + Z_{k,1} \tag{3.39}$$

where, the subscripts i and j refer to the incoming channels at a junction and the the subscript k refers to the outgoing channel. n_i and n_j refer to the last nodes on channels i and j, respectively. Equation (3.38) refers to the mass balance equation and Eq. (3.39) refers to the energy balance equation. It should be noted here that the energy losses at a junction and the velocity heads are neglected in Eq. (3.39). However, these assumptions can be easily relaxed if required. Solution for the IVP is applied successively for the converging networks as described below. Channel network shown in Fig. 3.1 is considered for the purpose of illustration.

1. Discharge in channel [7] is equal to the total discharge in all the root channels i.e.,$Q_1 + Q_2 + Q_4 + Q_5$. Flow depth at the node 8 is known either from the specifed boundary condition or from the rating curve at that section.

2. IVP is solved for channel [7] using the method described in section 3.4 and, $h_{4/7}$, the flow depth at node 4 in channel [7] is determined.

3. Equation (3.39) is used to determined the flow depths $h_{4/3}$ and $h_{4/6}$ in channels [3] and [6] respectively. In channel [6] i.e., Q_6 is equal to the total discharge in all the root channels feeding to it i.e.,$Q_6 = Q_4 + Q_5$. Discharge in channel [3], $Q_3 = Q_7 - Q_6$.

4. IVP is now applied to channels [3] and [6] to determine $h_{2/3}$ and $h_{6/6}$, respectively.

5. Step-3 is applied to nodes 2 and 6 to determine $(h_{2/2}, Q_2)$; $(h_{1/2}, Q_1)$; $(h_{6/4}, Q_4)$; and $(h_{6/5}, Q_5)$

6. IVP can now be applied to channels [1],[2],[4] and [5] to complete the water surface profile computation in the whole network.

The above discussed procedure is used to develop a computer code for determining water surface profiles in general, converging tree-type channel networks,given (1) the inflow discharges in all the root channels, (2) the flow depth at the pendant node, (3) bed elevation variation in all channels and, (4) variation in channel section properties in all channels. This is used as a subroutine in the aggradation or degradation model for the converging channel networks. The subroutine presented in this section itself calls the procedures presented in sections 3.2, 3.3 and 3.4 as subroutines.

3.5.2 Diverging Tree-type Channels Networks

In a typical diverging tree-type channel network (Fig. 3.2), flow from a root channel gets distributed to several branches. Such networks are common in (1) irrigation systems and, (2) river systems in deltaic region. For the case of simplicity ,it is assumed that (1) discharge from a single source enters the channel at upstream end, (2) there are no hydraulic structures within the system, (3) there are only two branches to a parent channel and, (4) the flow is subcritical through out the system.

However, the principles presented in this study can be easily extended to networks with multiple source and intermediate control structures. They can also be extended to cases where there are more than two branches to a parent channel. The problem of GVF computation in diverging tree-type channel systems can be defined as

"Given the discharge in the root channel (Q_2 in Fig. 3.2) and the controlling depths at all the pendant nodes($h_4, h_5, h_8, h_9, h_{11}$ and h_{12} in Fig. 3.2), compute the water surface profile in the system. Variation in the bed level and the channel shape parameters are also specified for all the channels." Successive application of the IVP as described in section 3.5.1 for this case would involve a trial and procedure because the discharge in the pendant channels is not known apriori. In this study, an efficient algorithm developed by Naidu et al. (1997) is used to compute the GVF profile in a diverging tree-type channel network. Details of this algorithm are presented elsewhere (Naidu 1995 Naidu et al. 1997) and only a brief description is presented here for the sake of completeness. The network shown in Fig. 3.2 is considered for the purpose of explanation.

The first step in the preposed method is to assume the discharge at a pendant node, say node 12. With this, the discharge and the controlling depth at the downstream end for channel [12] are known. Ttherefore, Eq. (1) can be solved for channel [12] as an "Initial Value Problem" (IVP) by marching the computations from node 12 to node 10. The Fourth-Order Runge-Kutta method is used for this purpose. At junction node 10, Eq. (3.39) can be applied to determine the upstream depth in channel [11] since the upstream depth in channel [12] is known from previous computations. With this, both upstream and down stream depths, i.e., at nodes 10 and 11 respectively are known for channel [11]. However, the discharge is not known. Therefore, Eq. (1) is solved for channel [11] as a "Boundary Value Problem" (BVP) for a single channel using the "shooting method" (Press et al.1986). In this method, the BVP is solved as an IVP with iterations.This computation determines

the discharge in channel [11] as a part of its solution. The continuty equation, Eq. (3.38) and the energy equation, Eq. (3.39) can now be applied at node 10 for determining the discharge and downstream depth for channel[10]. Computations can then be marched in channel [10] from node 10 to node 6 as an IVP, Eq. (3.39) can be applied to determine the u/s depth of channel [7]. However, neither the discharge nor the downstream depth in channel [7] are known. Therefore, computations at junction node 6 are different from the computations at the junction node 10. In this case, the group of channels [7], [8] and [9] are considered simultaneously. For the this subsystem, flow depths at nodes 6,8 and 9 are known. Therefore,it can be solved as a BVP for a group of channels. The computations are carried out as follows.

- Assume discharge in channel [8]
- Apply IVP in channel [8]
- Apply junction conditions at node 7
- Apply BVP in channel [9]
- Apply IVP in channel [7]
- If the computed depth at node 6 is same as the previously determined flow depth at this node, the assumed discharge in channel [8] is correct. Otherwise, it is updated using the "Shooting Method".

The above computations give the discharge in channel [7]. Equations (3.38) and (3.39) can now be applied at node 6 to determine the discharge and downstream depth in channel [6]. An IVP can be solved now to march the computations in channel [6] from node 6 to node 2. It can be seen from Fig. 3.2 that a BVP for a group of channels has to be solved at node 2 in order to determine the discharge in channel [3]. If this computed discharge in channel [2] is sum of the discharges

in channels [6] and [3]. If this computed discharge in channel [2] is same as the prescribed discharge at the root, the sssumed value of discharge in channel [2] is corerect. Otherwise, it is Updated using, once again the the shooting method and the computations are repeated till convergence. The flow chart for the above procedure with reference to the example network (Fig. 3.2) is shown in Fig. 3.3

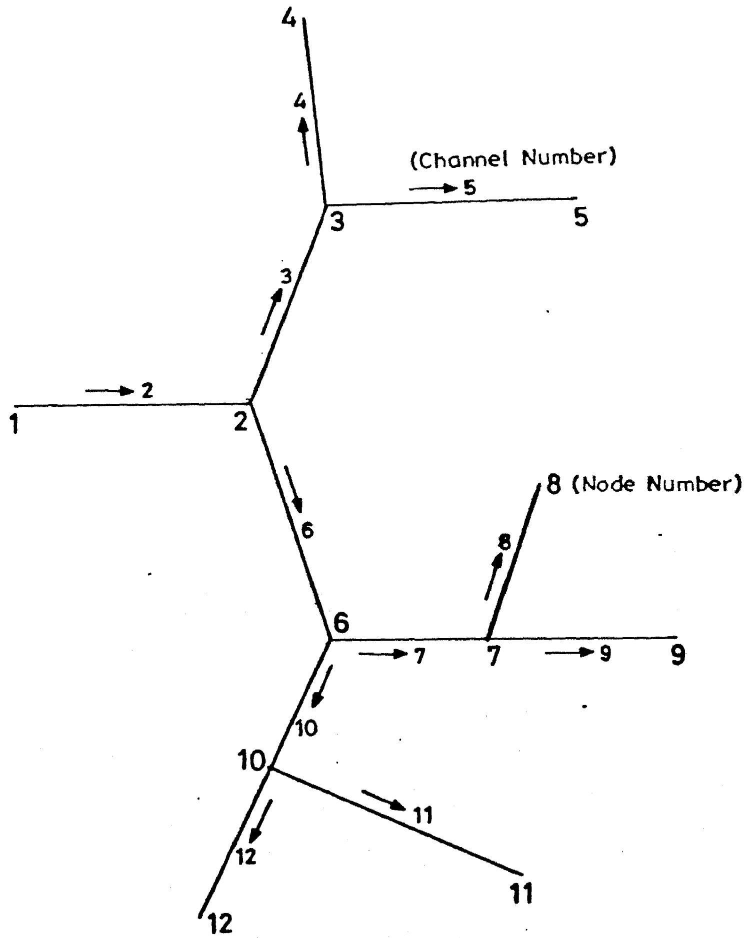


Figure 3.2: Typical diverging tree-type network

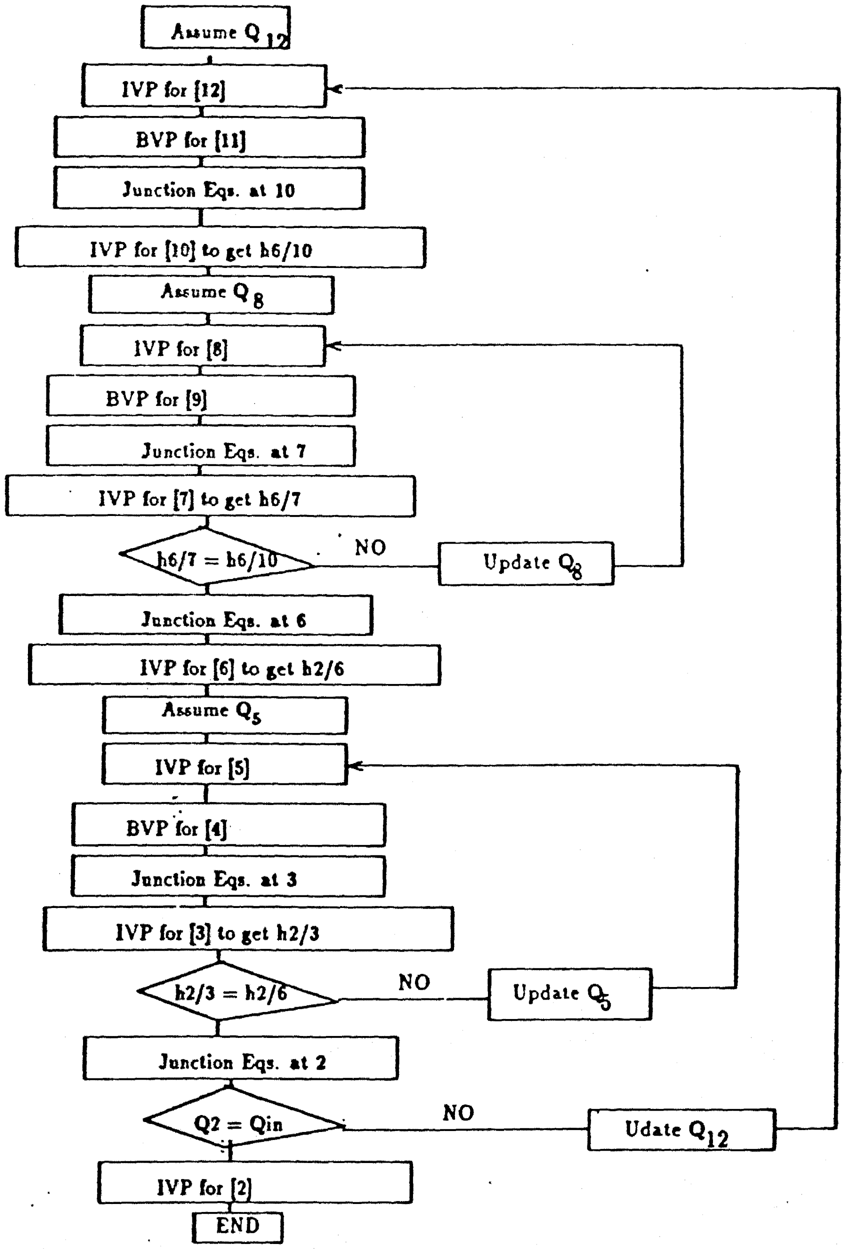


Figure 3.3: Flow chart for GVF Profile in diverging tree-type network

It is obvious from the above procedure that different "Paths of Marching" for the solution may be obtained depending on the pendant node at which the computations are started. The path of marching also depends on node chosen for starting the iterations of "Boundary Value Problem for Group of Channels, BVPGC". The proposed algorithm results in high efficiency when calculations march along a particular path in the network. For example, consider the case of starting the computations at the pendant node 5 for the network shown in the Fig. 3.2. This choice would result in a six fold increase in the computational time as compared to the previous choice of starting the computations at node 12. Naidu et.al. (1997) presented an algorithm for the best path of marching using some simple graph theoritic principles.

The computer code developed by Naidu et al. (1997) is used in this study to compute water surface profiles in diverging tree type channel networks. The original code developed by Naidu et al. (1997) used a uniform slope for each channel and considered only cross-sections of trapezoidal shape. This code is modified by the author to include variable bed slope and more general cross-sectional shapes as given by Eq. (2.2). The modified code is used as a subroutine in the aggradation/ degradation model for diverging tree-type channel networks.

3.6 Computation of Aggradation/Degradation

The aggradation/degradation at a channel section is computed by numerically solving the sediment continuity Eq. (2.6). While solving the Eq. (2.6), prior computed sediment discharge values from the water flow module are used. Backward spatial and backward time finite- differences are used to discretize Eq. (2.6). The change in the sediment flow area at any any section 'j' in channel 'i', $\Delta A_{s_{i,j}}$ is given by

$$\Delta A_{s_{i,j}} = \frac{1}{1-p}\frac{\Delta t}{\Delta x}\left[Q_{s_{i,j-1}} - Q_{s_{i,j}}\right] \tag{3.40}$$

or, for a wide channel,

$$\Delta Z_{i,j} = \frac{1}{1-p}\frac{\Delta t}{\Delta x}\left[q_{s_{i,j-1}} - q_{s_{i,j}}\right] \tag{3.41}$$

where, $\Delta t =$ computational time step. Equations (3.40) and (3.41) should not be applied at the upstream nodes of the root channels and at the junction nodes. Boundary conditions are applied at the upstream nodes of the root channels. Junction nodes are treated in a special manner to ensure sediment mass balance condition.

3.6.1 Upstream Boundary Condition

Inflow sediment discharge hydrographs are specified as the boundary conditions at all the upstream nodes i.e. j=1 in all the root channels. $\Delta A_{s_{1,i}}$ is determined using the following equation.

$$\Delta A_{s_{1,i}} = \frac{1}{1-p}\frac{\Delta t}{\Delta x}\left[Q_{s_{i,0}} - Q_{s_{i,1}}\right] \tag{3.42}$$

where $Q_{s_{i,0}}$ refers to the sediment inflow at time $t+\Delta t$ for channel 'i' and is obtained from specified inflow sediment discharge hydrograph. A similar procedure was used by Bhallamudi and Chaudhry (1992) and satisfactory results were obtained for the case of aggradation due to sediment overloading.

3.6.2 Aggradation/Degradation of Junction nodes

The procedure for determining the aggradation/degradation at a junction node is slightly different from Eq. (3.40) for an interior node. Refering to Fig. 3.4,

$$\Delta Z_{jun} = \frac{1}{1-p}\Delta t\left[\frac{Q_{s_{i1,n_{i1}-1}} + Q_{s_{i2,n_{i2}-1}} - Q_{s_{i3,2}}}{B'_{i1}\,\Delta x_{i1} + B'_{i2}\,\Delta x_{i2} + B'_{i3}\Delta x_{i3}}\right] \tag{3.43}$$

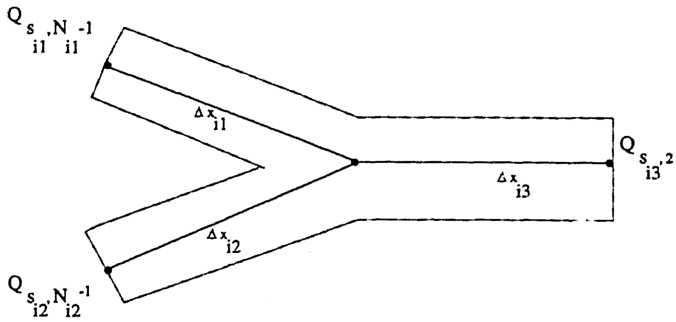


Figure 3.4: Schematic layout of a junction

where, ΔZ_{jun} = change in the bed level at the junction in time Δt. Subscripts $(i1, n_{i1} - 1)$ and $(i2, n_{i2} - 1)$ refer to penultimate nodes in channel i1 and i2, respectively. Subscript (i3,2) refers to the second node in channel i3. Subscripts i1,i2 and i3 refer to the channels i1,i2 and i3 respectively. B' refers to an "effective bottom width" for the channel. This concept is necessary to avoid singularity problem in case of channels of zero bottom width such as, the triangular channels. At the present stage of code development, B' is taken as the bottom width B_o. Numerical simulations are carried out only for those cases where all the channels have finite bottom width. It can be seen that Eq. (3.43) is an expression for sediment mass balance condition for a control volume sarrounding the nodes $(i1, n_{i1}-1)$, $(i2, n_{i2}-1)$ and $(i3, 2)$. An inherent assumption in Eq. (3.43) is that the bed aggrades/ degrades by equal amounts on all the three sides of a channel junction. This assumption is questionable in case of junctions with free-overfalls and needs further study.

It should also be noted here that the aggradation/degradation at the last node of any pendant channel is computed also using Eq. (3.40). The inherent assumption in this is that the outflow sediment discharge from a pendant node is equal to sediment-discharge capacity at the last node of the channel. This assumption is generally valid for the test cases presented in the study. However, it may not be strictly valid in some other cases. For example, if the downstream control structure is a spillway, it

may allow water to flow over, but reduces sediment outflow to zero.

3.6.3 Change in Cross-Section

Aggradation/degradation at a cross-section will result in change in the bed level as well as the cross-sectional shape. For wide channels, these changes are completely described by Eq. (3.41) since there would not be any appricable change in the bank configuration. For non-rectangular channels, however, the value of $\Delta A_{s_{i,j}}$ at any cross-section should be used to update the bed levels and the channel shape parameters.

Aggradation: In the case of aggradation, it is assumed that the banks would not be able to support the deposited material and deposition during the time Δt occurs completely on the bed (Figure 3.5a). Change in the bed level, ΔZ is computed inversely from the following equation.

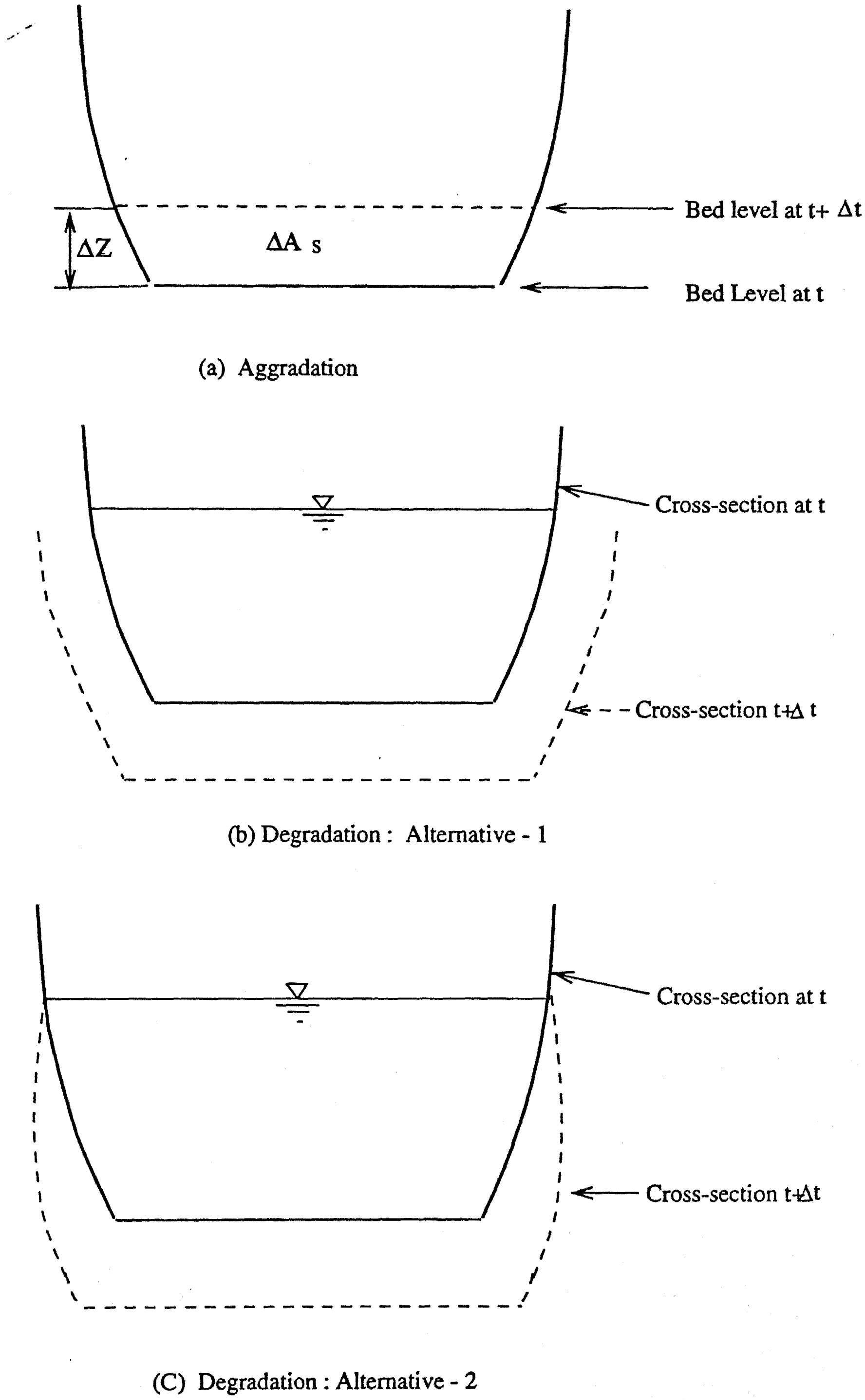
$$\Delta A_s = B_0 \, \Delta Z + \frac{C_1}{1+C_2}(\Delta Z)^{1+C_2} \frac{C_3}{1+C_4}(\Delta Z)^{1+C_4} \tag{3.44}$$

Updated value of B_0 is determined using the equation

$$B_0^{new} = B_0^{old} + C_1(\Delta Z)^{C_2} + C_3(\Delta Z)^{C_4} \tag{3.45}$$

Values of B_l and B_r are accordingly changed and new values of C_1, C_2, C_3 and C_4 are determined using Eqs. (3.3) and (3.4). Slight deviations in the bank profile may be observed because of the statistical procedure adopted in determining C_1, C_2, C_3 and C_4.

Degradation: Degradation is assumed to occur due to erosion from the entire wetted perimeter. Two alternative procedures are adopted to compute the new cross-section due to degradation. In the first alternative (Fig. 3.5b), a parallel degradation is assumed, *i.e.*, the entire cross-section expands uniformly by an amount ΔZ. ΔZ in this case is given by



(a) Aggradation

(b) Degradation : Alternative - 1

(C) Degradation : Alternative - 2

Figure 3.5: Aggradation/Degradation in a non-rectangular channel

$$\Delta Z = \frac{\Delta A_s}{P} \tag{3.46}$$

B_0, $B_l(y)$ and $B_r(y)$ values are occordingly changed and new values of C_1, C_2, C_3 and C_4 are determined using Eqs. (3.3) and (3.4).

In the second alternative, the degradation at any point on the banks or the bed depends on the local shear stress. The local shear stress τ_l' is assumed to be given by (Chang and Hill 1982)

$$\tau_l' = \rho\, g\, h'\, S_f\, cos\theta \tag{3.47}$$

where, h' = local flow depth and θ = local transverse slope of the bank.

The decrease in area will be distributed such that the local scour depth (Z_s) is proportional to the local shear stress. The local scour depth can be obtained by (Chang and Hill 1976)

$$Z_s = \frac{\tau_l' - \tau_{cr}}{\sum_T [\tau_l' - \tau_{cr}]\, \Delta y} \Delta A_s \tag{3.48}$$

where, τ_{cr} = critical shear stress, Z_s = local scour depth Δy = the horizontal distance between 2 points and T = top width of the channel. If the shear stress is negitive then it is set to zero. The new values of C_1, C_2, C_3 and C_4 are determined using Eqs. (3.3) and (3.4).

3.7 Quasi-Steady Flow Uncoupled Algorithm

The principles presented in sections 3.1 through 3.6 are used to develop quasi-steady flow uncoupled models for aggradation/degradation in alluvial channel networks. the flow chart of the algorithm is presented in Fig. 3.6.

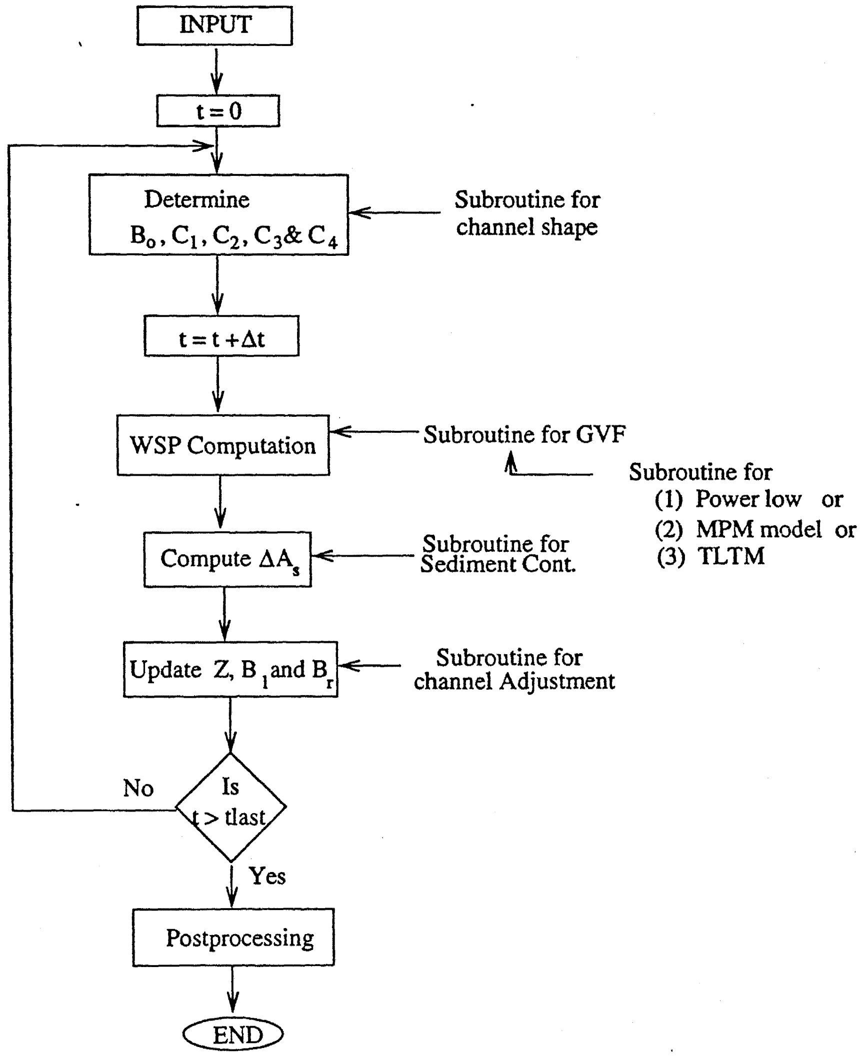


Figure 3.6: Flow chart for the quasi-steady flow model

3.8 Closure

In this chapter, details of quasi–steady flow uncoupled models for computing aggradation and degradation in tree-type alluvial channel networks are presented. It is envisaged that these models will be eventually used for simulating bed level changes in networks as large as the Ganga river system, and for periods as long as 20 years. Therefore, every effort is made is to reduce the computational cost per one computational time period while formulating these models.

Chapter 4

Results And Discussion

In this chapter, the quasi-steady flow, uncoupled model is verified by comparing the present numerical results with (1) observed data (Soni et al. 1980) and earlier numerical results (Bhallamudi and Chaudhry 1991) obtained using a complete unsteady, coupled model. The applicability of the numerical model to converging and diverging tree-type alluvial channel networks is demonstrated by simulating (1) aggradation due to sediment overloading, and (2) degradation due to sediment depletion. Results are presented for both rectangular channels and nonrectangular channels.

4.1 Verification of The Model

The present numerical model was used to simulate the aggradation process observed by Soni et al. (1980) in a laboratory flume due to sediment over loading at the inflow section. Soni et al. (1980) used a rectangular flume with alluvial bed carrying a constant unit discharge, q_0, at a uniform flow depth of h_0. The equilibrium between the water flow and the sediment discharge was distrubed by increasing the sediment discharge at the upstream end from its equlibrium value of q_{s0} to $q_{s0} + \Delta q_s$. This

resulted in the aggradation of the flume bed so that the increased bed slope could transport the additional imposed sediment load that deposited on the bed.

The experiments were conducted in a flume 0.2m wide and 30m long. The sand forming the bed and the injected meterial had a mean diameter, d_{50} of 0.32mm. The values of empirical constants a and b in the sediment transport formula Eq. (2.9) as found from the uniform flow experiments were 1.45×10^{-3} and 5.0, respectively. Manning's roughness coefficient 'n' was approximatly equal to 0.022 and the porosity 'p', of the sediment bed layer was equal to 0.4.

In the numerical simulation, the values of bed slope $s_0 = 0.0026$, unit discharge, $q_0 = 0.032\,m^2/s$, flow depth, $h_0 = 0.075$ m and, sediment load increment,$\Delta q_s = 2.7 q_{s0}$. These data were taken from the run identified as test no.2 by Bhallamudi and Chaudhry (1991). The initial bed elevations at all finite difference nodes as calculated from s_0 were specified as the initial conditions. Soni et al. (1980) conducted the experiments in a single channel. However, a three channel system as shown in Fig. 4.1 was considered for the application of the numerical model. The width of the pendant channel [3] was adjusted(B_3=0.4m) such that the flow conditions in the two root channels [1] and [2] were almost same as the flow conditions obtained by Soni et al. (1980). Lengths of the channels were 30m each, with the total number of reaches in a channels being 30 ($\Delta x = 1.0$m). The computational time step, Δt was taken as 1.0 sec. The uniform flow depth was specified as the downstream boundary condition and sediment loads $q_{s0} + \Delta q_s$ were specified as the two upstrem boundary conditions for the root channels [1] and [2].

The numerical run was made for a total time, t=3000sec. The bed disturbance i.e., the aggradation profile, from the root channels did not reach the confluence point during this period of computation. Therefore, the aggradation characteristics in the two root channels in the numerical simulation were close to those in Soni et al. (1980).

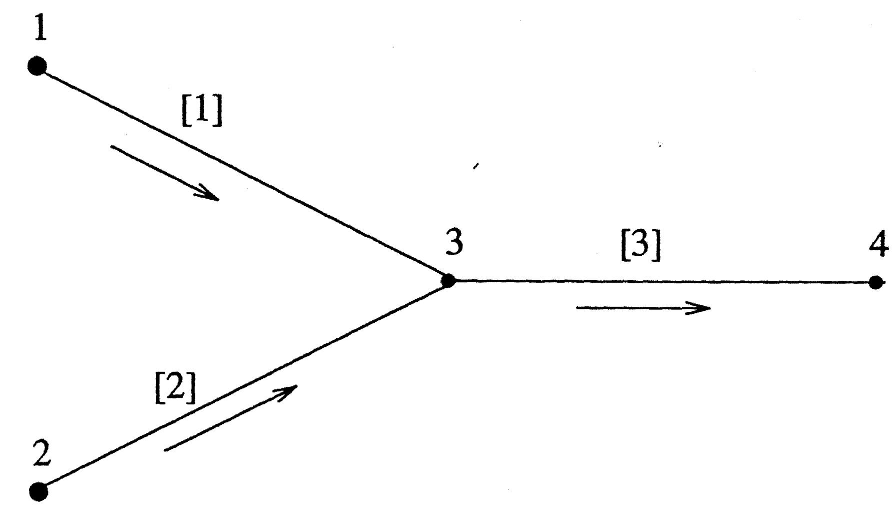


Figure 4.1: Three channel system

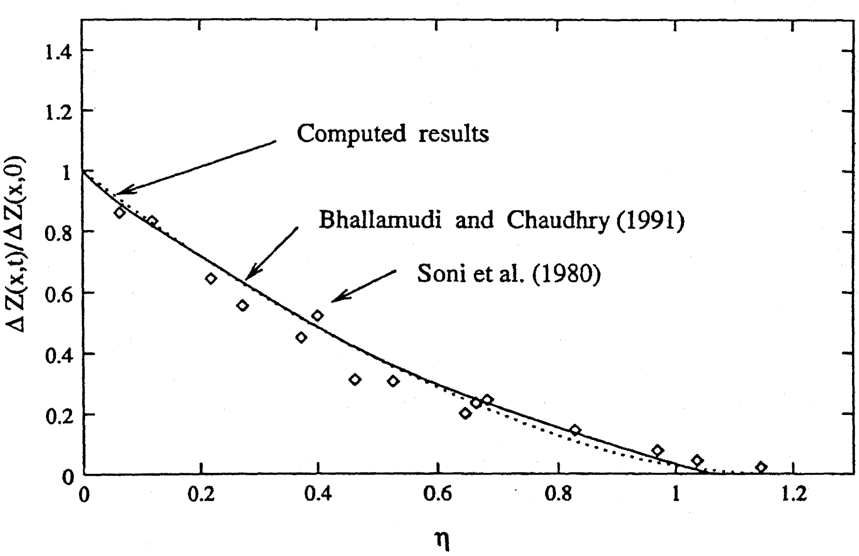


Figure 4.2: Variation of relative aggredation with η, sediment overloading

Figure 4.2 shows the variation of the relative aggradation, i.e., $\Delta Z(x,t)/\Delta Z(0,t)$ with the non-dimensional parameter, $\eta = x/2\sqrt{K_0 t}$, where K_0 is the diffusion coefficient which is equal to $b\,q_{s0}/3s_0(1-p)$. Numerical results obtained using the present model are compared in Fig. 4.2 with the measured data (Soni et al. 1980) and with the numerical results obtained by Bhallamudi and Chaudhry (1991) using a complete unsteady coupled model. It can be observed from Fig. 4.2 that the non dimensional bed profiles obtained by the present quasi-steady flow model compared very well with the results obtained by Bhallamudi and Chaudhry (1991) who used a complete one dimensional partial differential equations for un-steady flow in a wide rectangular channel with deformable bed. This satisfactory matching indicates that the quasi-steady flow assumption and the uncoupled solution technique do not introduce any significant errors when simulating bed level changes. This aspect of modeling was confirmed also by Cui et al. (1996). However, the quasi-steady flow model may break down if the Froude number for the flow is close to unity. Figure 4.2 is drawn for aggradation in root channel [1]. Identical results were obtained for the other root channel [2] as expected. The bed wave had not reached the junction during the period of computation. Therefore, the bed profile in the pendant channel [3] did not change as expected.

4.2 Aggradation Due to Sediment overloading

The objective of this work is to develop a numerical model for aggradation in genaral diverging and converging tree-type networks which considers genaral nonrectangular cross-sectional shapes and a sophisticated predictor for computing sediment transport capacity. To demonstrate its applicability, several test runs were made, results of which are discussed in this section and the following section . This section presents the results for aggradation due to sediment over loading.

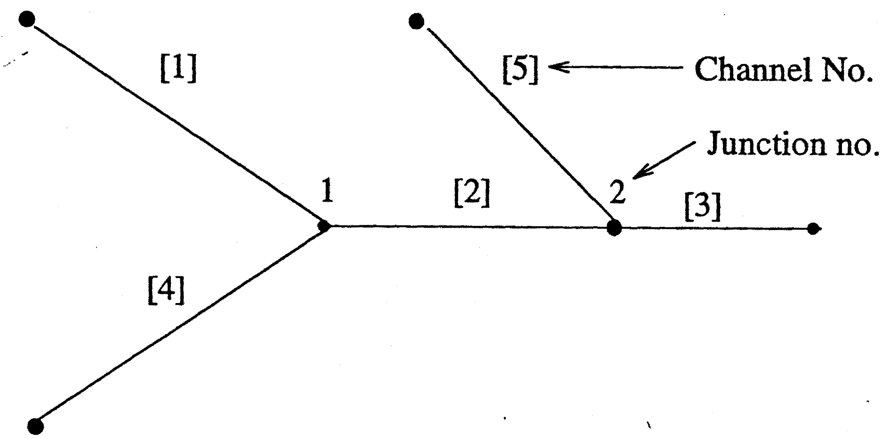


Figure 4.3: Five channel system, sediment overloading

4.2.1 Network With More Than One Junction

In the previous section, simulations were made for a three channel system with one junction point. However, in a general network there can be more than one junction. Simulations for Scenarios (1) and (2) in this study were made for a five channel system of rectangular channels as shown in Fig. 4.3. The Power law model and the Iowa model for sediment discharge were used in Scenarios (1) and (2), respectively. The lengths of all the five channels were taken as 30m. An initial bed slope s_0=0.0026 was taken for all the five channels. The values of bed elevations as calculated from s_0 were assigned to each finite difference node as initial conditions. The constants a and b in Eq. (2.9) were taken as 1.45×10^{-3} and 5.0 respectivly. The discharge per unit width in all the channels was taken equal to $0.032m^2/s$. Mannings's roughness coefficient in all channels was 0.22. The width of each root channel was equal to 0.2m. Widths of other channels in the system were taken such that the unit discharge was constant through out the system. Sediment overloading was taken as 3.7 times the initial sediment carrying capacity at the upstream most node in all the root channels. Porosity of the bed layer was taken as 0.4.

The model was run for 3000sec with Δt=1.0sec and Δx=1.0m. Figures 4.4 and 4.5 show the initial bed profile and the final bed profile at t=3000 sec. for channels [1] and [2], respectively.

The bed wave from the root channels had not reached any of the junction points. Therefore, there should not be any change in the bed profile in the channels other than the root channels i.e.,channels [2] and [3] in Fig. 4.3. The bed profiles for the channel [2] shown in Fig. 4.5 confirms that the model gives consistent results. The bed profiles for all the root channels were identical to each other as expected. The bed profiles for channels [1] are presented in Fig. 4.4. It is obvious from this figure that the bed wave had not reached the junction point.

In the simulation for the Scenario 2, the initial and other conditions were identical to those in Scenario 1, except the model for the sediment carrying capacity. The Iowa model was used to compute s_f and q_s in this case. The mean size of the sediment particles, d_{50}=0.32mm. Figures 4.6 and 4.7 shows the initial bed profile and the final bed profile at t=3000sec in channel [1] and [2] respectivly. AS expected there was no aggradation in channel [2] Fig. (4.7) because the the bed wave had not reached the junction points. One can not compare the results obtained for Scenario's 1 and 2 because two different empirical models were used for computing the sediment carrying capacity. However, the constiency in the results can be checked. It can be observed from Figs. 4.4 and 4.6 that the aggraded more in the case of Scenario 1. This is because the sediment carrying capacity and consequently the sediment overloading is more if the power law is used. Sediment carrying capacity at the upstream most section, q_{s0} is equal to $1.75 \times 10^{-5}\, m^2/s$ if Eq. 2.9 was used, while it is equal to $7.25 \times 10^{-6}\, m^2/s$ if Eqs. 2.14 and 2.15 were used.

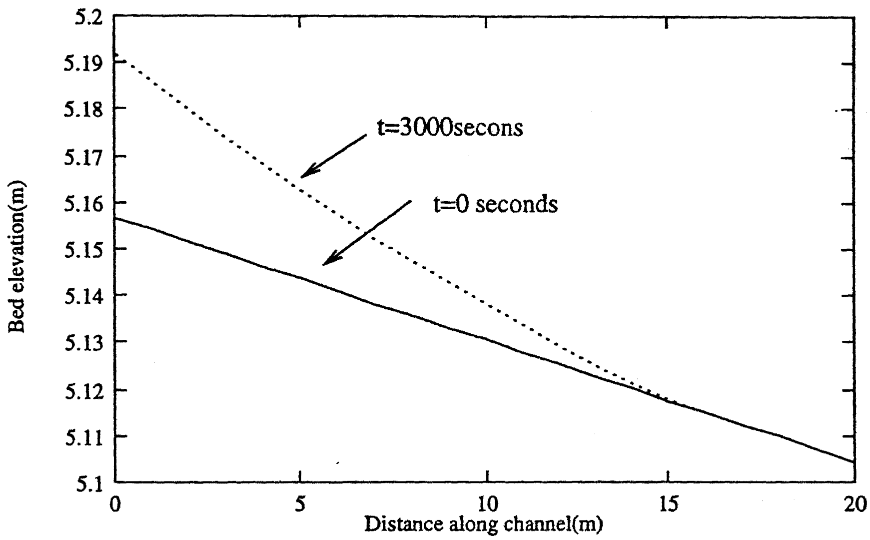


Figure 4.4: Bed profile in Channel 1, Scenario-1

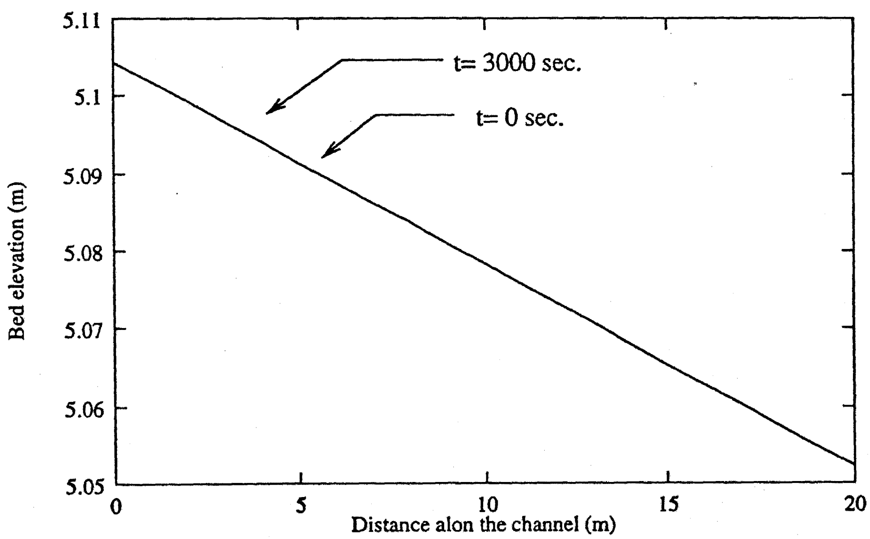


Figure 4.5: Bed profile in Channel 2, Scenario-1

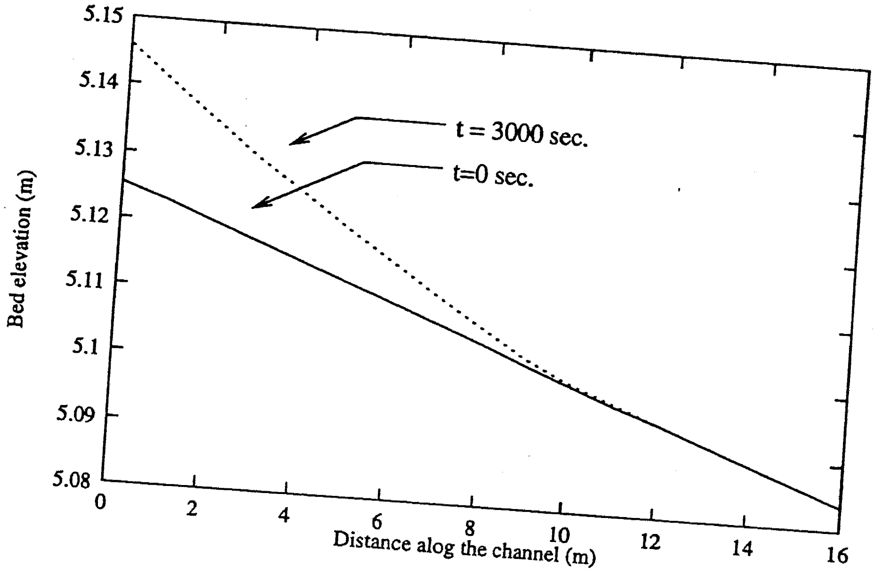


Figure 4.6: Bed profile in Channel 1, Scenario-2

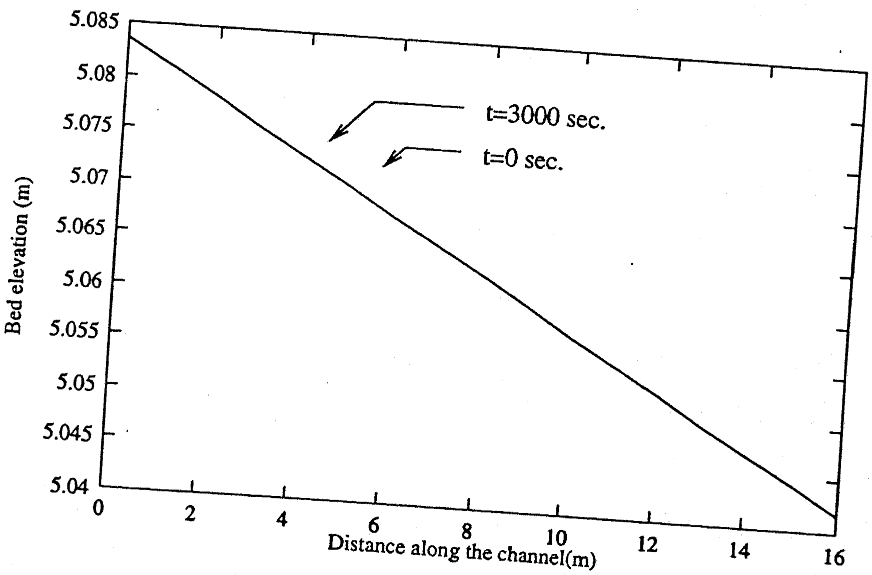


Figure 4.7: Bed profile in Channel 2, Scenario-2

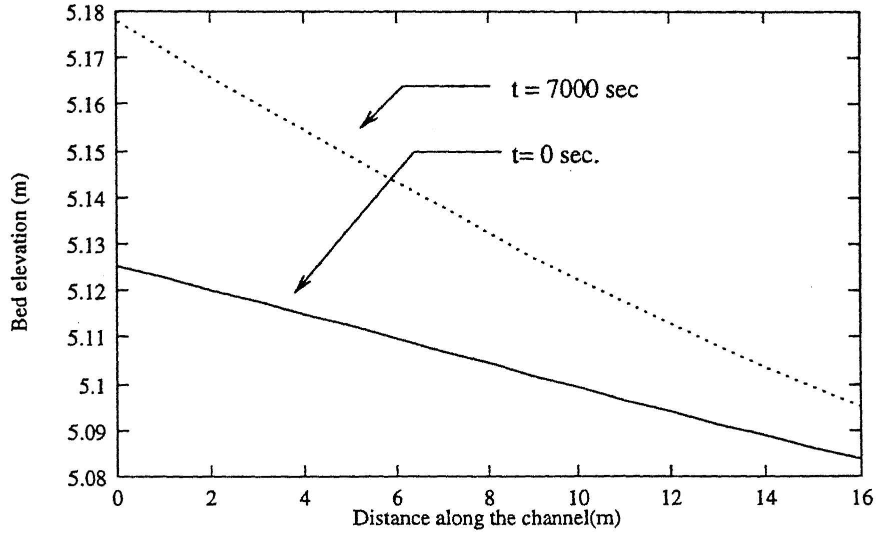


Figure 4.8: Bed profile in Channel 1, Scenario-3

4.2.2 Aggradation Beyond The Junction points

In Scenarios 1 and 2, it was observed that the bed wave had not reached the junction points. The simulation for Scenario 3 was performed to demonstrate the applicability of the model when the bed wave propagates beyond the junction point. The test conditions for the Scenario 3 were identical to those in Scenario 1 except that the lengths of the channels were 17.0m in this case. Also, the computations for the Scenario 3 were performed for a total time of 7000 sec.

Figures 4.8 to 4.12 shows the initial bed profile and the bed profile after 7000 seconds in channels [1],[2],[3] ,[4] and [5] respectly. It can be observed from Figs. 4.8 to 4.12 that the aggradation had occured in all the channels. This indicates that the bed wave had moved beyond the junction points. No numerical difficulties were encountered while performing the computations. It can be observed from Fig. 4.9 that there is a sharp rise in the bed level change at the downstream end of the channel [2]. This is because the bed wave from the channel [5] arrived at the junction 2 earlier than the disturbance from the upstream side. The resulting

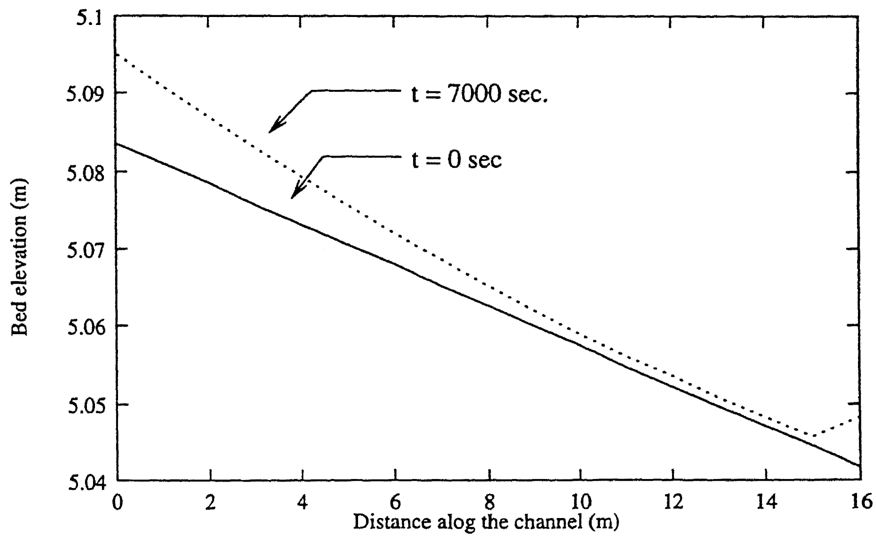


Figure 4.9: Bed profile in Channel 2, Scenario-3

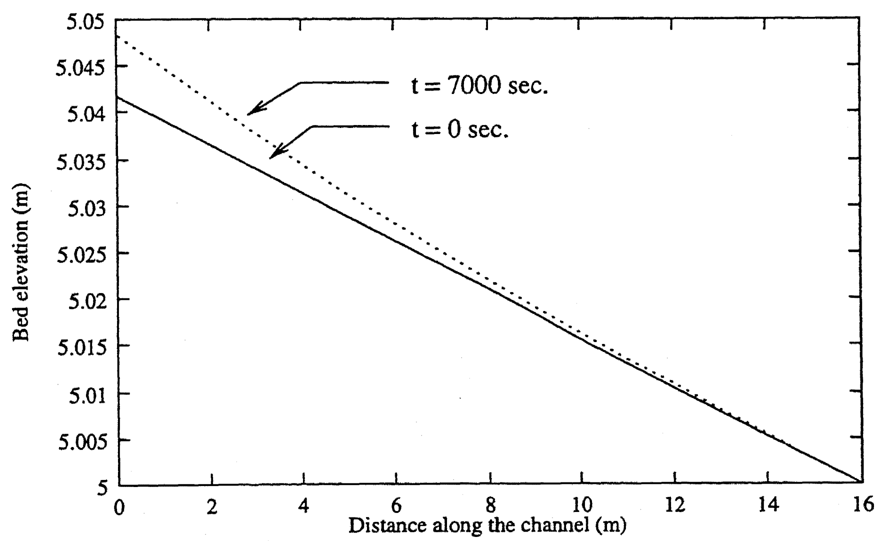


Figure 4.10: Bed profile in Channel 3, Scenario-3

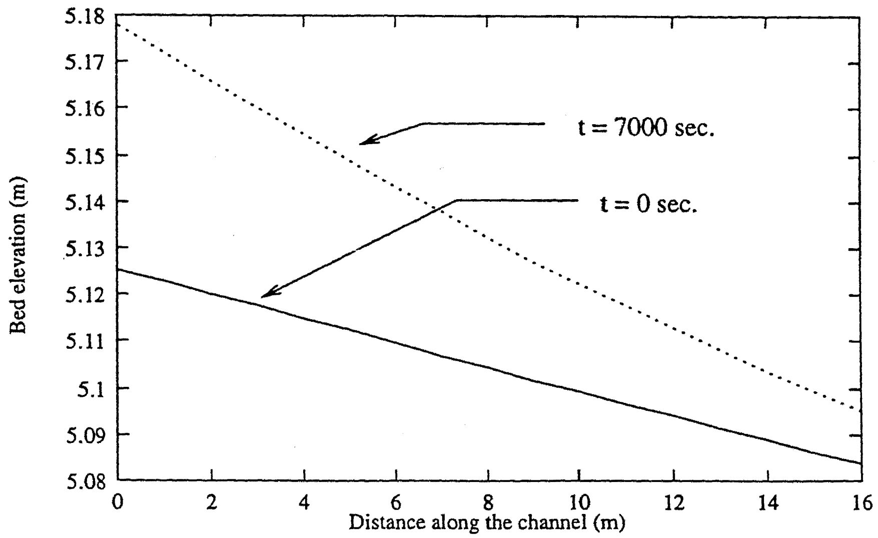


Figure 4.11: Bed profile in Channel 4, Scenario-3

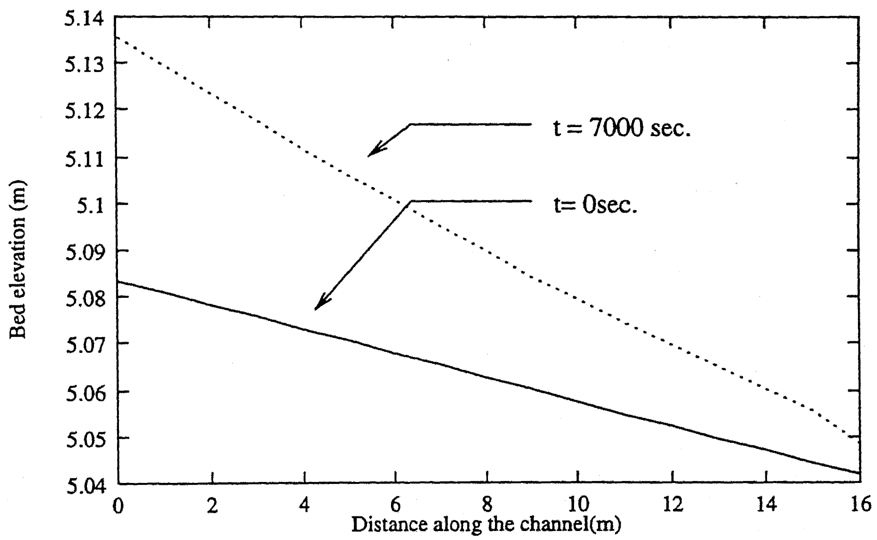


Figure 4.12: Bed profile in Channel 5, Scenario-3

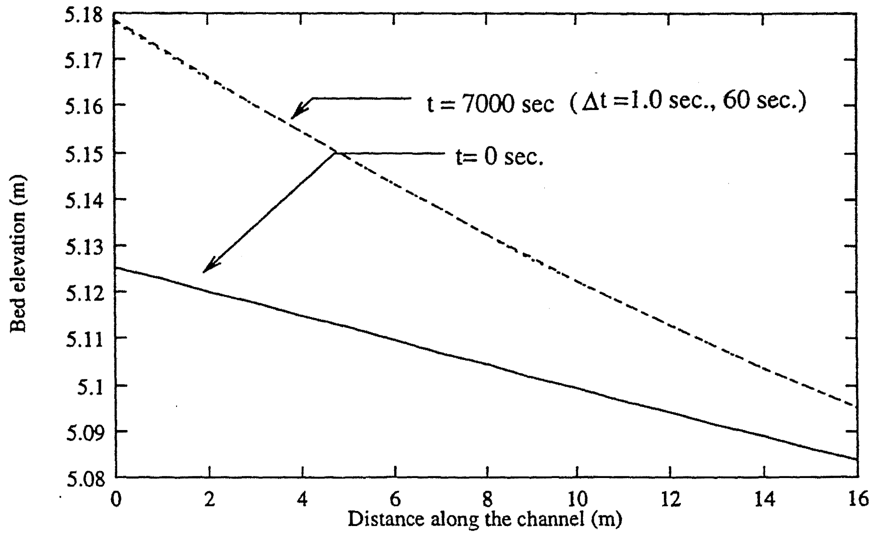


Figure 4.13: Bed profile in Channel 1, Scenario-4

aggradation moved in an upstream direction in channel 2 toward junction 1. This occured simultationeously with the movement of aggradation in the downstream direction from junction 1 to 2.

4.2.3 Effect Of Computational Time Step

In the simulation for the Scenarios 1, 2 and 3 the computational time step, Δt was equal to 1.0 sec. Simulation for the Scenario 4 was performed to demonstrate that the results can be obtained with a larger Δt without introducing significant error. In the Scinario 4 the test conditions were identical to Scenario 3, except that Δt=60sec.Figures 4.13-4.17 show the initial bed profile, the bed profile after 7000 sec with Δt=1.0sec and, the bed profile after 7000sec with Δt=60sec. It can be observed from these figures that even when Δt was taken as large as 60 times the previous value, the difference in the aggradation profiles was negligible. This fact is important from the point of view of applicability of this model to larger networks. The courant number for stability in the application of a quasi-steady flow model

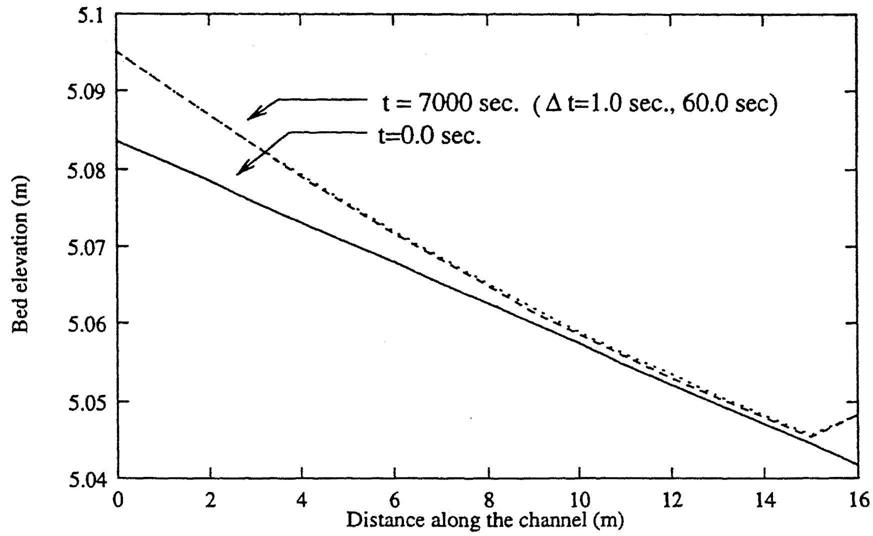


Figure 4.14: Bed profile in Channel 2, Scenario-4

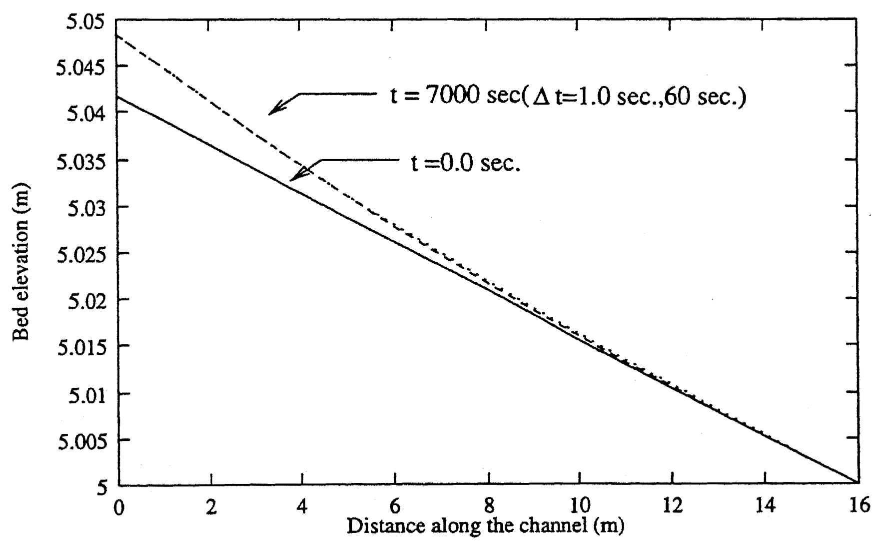


Figure 4.15: Bed profile in Channel 3, Scenario-4

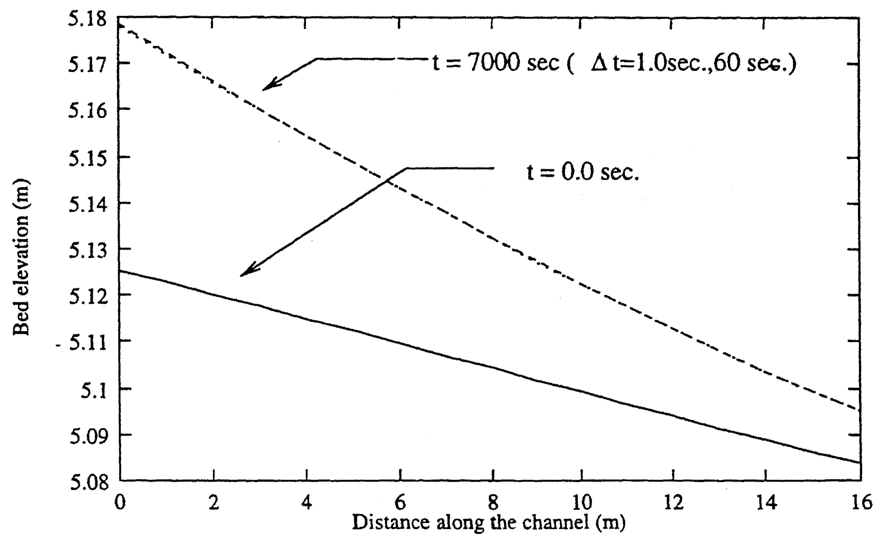


Figure 4.16: Bed profile in Channel 4, Scenario-4

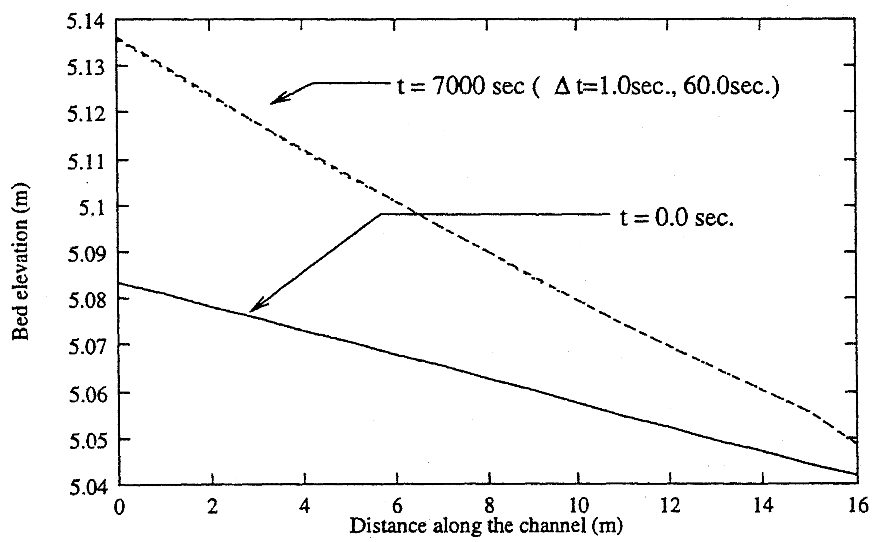


Figure 4.17: Bed profile in Channel 5, Scenario-4

Table 4.1: Cross-sectional shape parameters for Scenario-5

channel no.	Cross-sectional Shape Parameters				
	B_0(m)	C_1	C_2	C_3	C_4
1	0.2	1.00083	1.20098	1.00093	1.20098
2	0.51099	0.97138	1.19032	0.97138	1.19032
3	0.51099	0.97138	1.19032	0.97138	1.19032
4	0.2	1.00083	1.20098	1.00093	1.20098
5	0.2	1.00083	1.20098	1.00093	1.20098

depends on the bed wave velocity. The bed wave velocity is usually very small (Cunge et al. 1980) and therefore, larger Δt values could be used.

4.2.4 Aggradation Profiles In Non-rectangular Channels

In all the previous simulations, the chanels were of rectangular shape. To demonstrate the applicability of the present model for non-rectangular channels, simulation was performed for Scenario 5. Scenario 5 considers also a five channel converging network as shown in Fig. 4.3. The constants B_0, C_1, C_2, C_3 and C_4 for the cross-sectional shape for channels [1],[2],[3],[4] and [5] are given in the Table 4.1 In this test case, Iowa model was used for computing the sediment carrying capacity. The initial bed slopes S_0 were taken as 0.0012, 0.00082, 0.0021 for the root channels [1], [2] and [3], respectively. S_0 value of 0.0012 was taken for the channels [4] and [5]. The discharge in the root channels i.e.,[1],[4] and [5] was equal to $0.0062m^2/s$. Lengths of the channels were equal to 30m each. Particle mean size and the porosity were taken as 0.32mm and 0.4, respectively. Computations were made for 3000sec with Δt =1.0sec. Figures 4.18-4.19 show the change in the bed profile in the channel [1] and the change in the cross-sectional shape at the inflow section after 3000sec. Identical results were obtained for all the root channels as expected. In the numerical

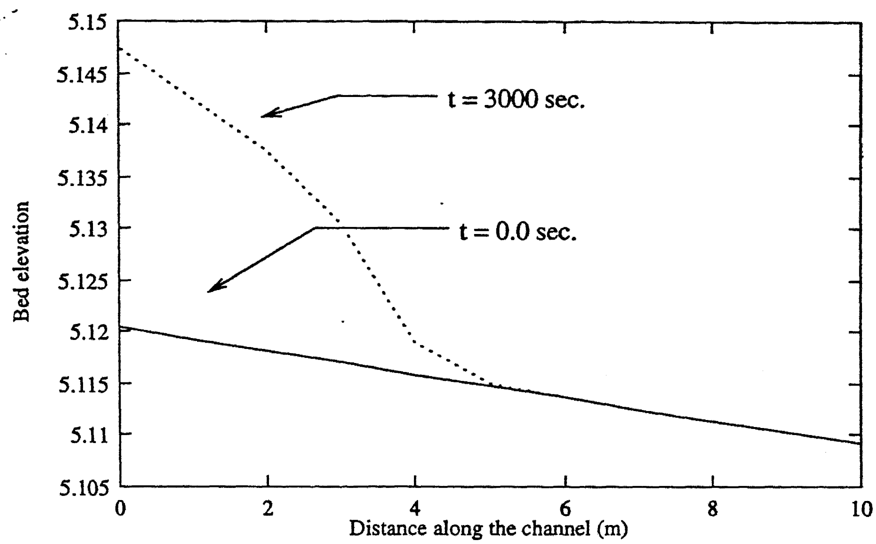


Figure 4.18: Bed profile in Channel 1, Scenario-5

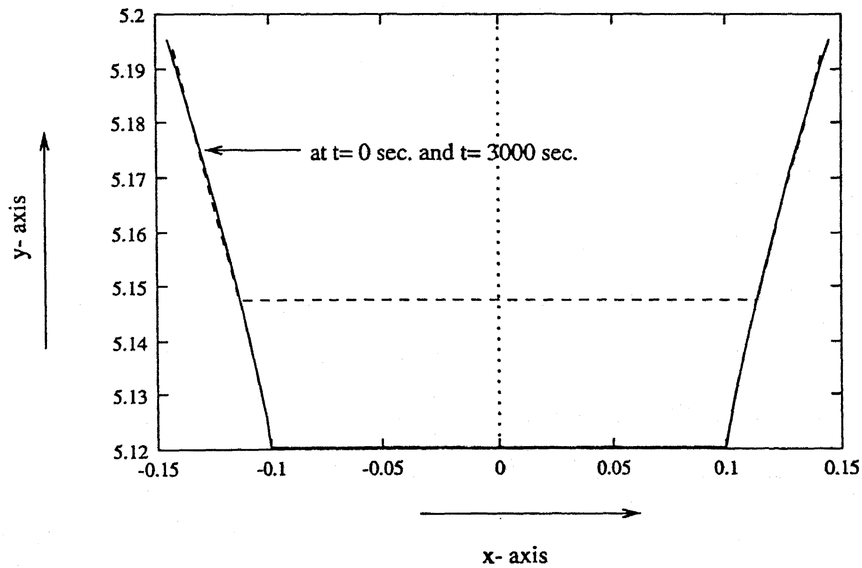


Figure 4.19: Temporal variation of cross section at inflow to Channel 1, Scenario-5

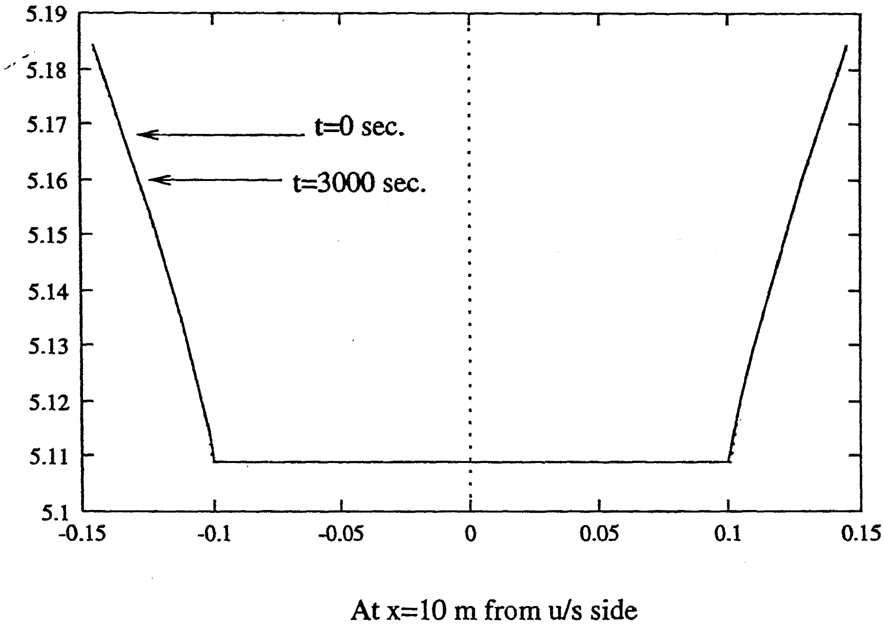


Figure 4.20: Temporal variation of cross section at 10m from U/S, Channel 1, Scenario-5

procedure described in section 3.6.3, it was assumed that all the deposition occurs on the bed and bed rises in parallel layers as a function of time (Chang and Hill 1976).

Results presented in Fig 4.19 confirm that the above procedure has been correctly adopted in the computer code. There was no aggradation beyond 8m (Fig. 4.18) because the bed wave had not arrived at this point within the computational time period. It should be noted here that the bed levels and the cross-section would not change until the bed wave arrives from the upstream side because the initial flow conditions were uniform through out the network. This is also reflected in Fig. 4.20 for the cross-section at x=10m.

Table 4.2: Cross-sectional shape parameters for Scenario-6

channel no.	Cross-sectional Shape Parameters					Initial bed slope
	B_0(m)	C_1	C_2	C_3	C_4	S_0
2	0.1	0.94934	1.7714	0.94934	1.7714	0.0031
3	0.1	0.94934	1.7714	0.94934	1.7714	0.00066
4	0.1	0.94934	1.7714	0.94934	1.7714	0.00066
5	0.1	0.94934	1.7714	0.94934	1.7714	0.00026
6	0.1	0.94934	1.7714	0.94934	1.7714	0.00026

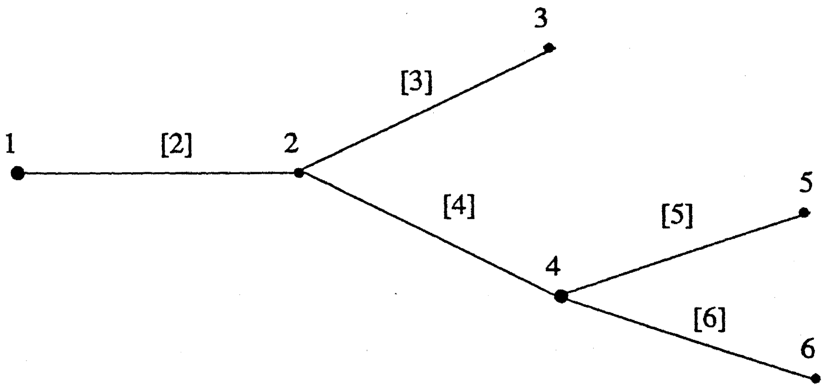


Figure 4.21: Five channel system, sediment depletion

4.2.5 Degradation Due To Sediment Depletion

In the proor discussed simulations, aggradation in a converging channel system was considered. Applicability of the model to diverging tree-type networks and to simulate degradation is demonstrated in this section. In the simulation for the Scenario 6, a five channel diverging tree-type network as shown in Fig. 4.21 was considered. Length of each channel was equal to 30m. The specified values of the initial sectional shape parameters B_0,C_1, C_2, C_3, C_4 and the initial bed slopes, S_0 are presented in Table 4.2. Porosity of the sediment bed layer was taken as 0.4.

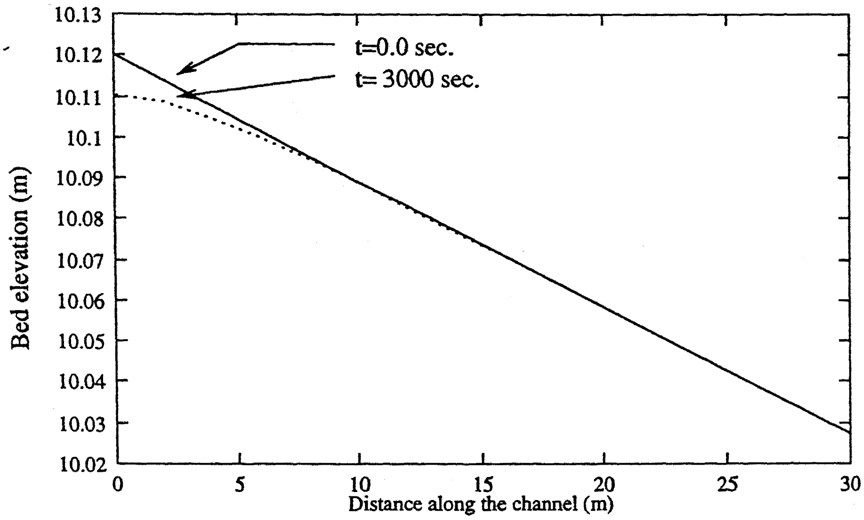


Figure 4.22: Bed profile along Channel 2, Scenario-6, Procedure 1

The mean size of the sediment particles and the specific gravity of the particle were taken as 0.32mm and 2.65 respectinely. A flow depth of 0.075m was specified as the downstream boundary condition for all the pendant channels i.e.,[6],[5] and [3]. Total discharge in the root channel [2] was taken as $0.0062m^3/s$. The sediment carrying capacity and the friction slope were computed using the Iowa model. Simulation for the Scenario 6 considers the degradation of the channel system due to complete depletion of the sediment load into the root channel [2] (Fig. 4.21). Simulation were performed for a total time =3000sec. with Δx=2m, and Δt=1.0 sec. While performing the simulations for the Scenario 6, two different procedures as discussed in chapter 3 were adopted for modelling the cross-sectional change due to degradation. In the procedure 1, the degradation occured without changing the cross-sectional shape while, in the procedure 2, the degradation was proportional to the local shear stress (Chang and Hill 1976)

Figure 4.22 shows the change in the bed profiles after 3000sec. for the root

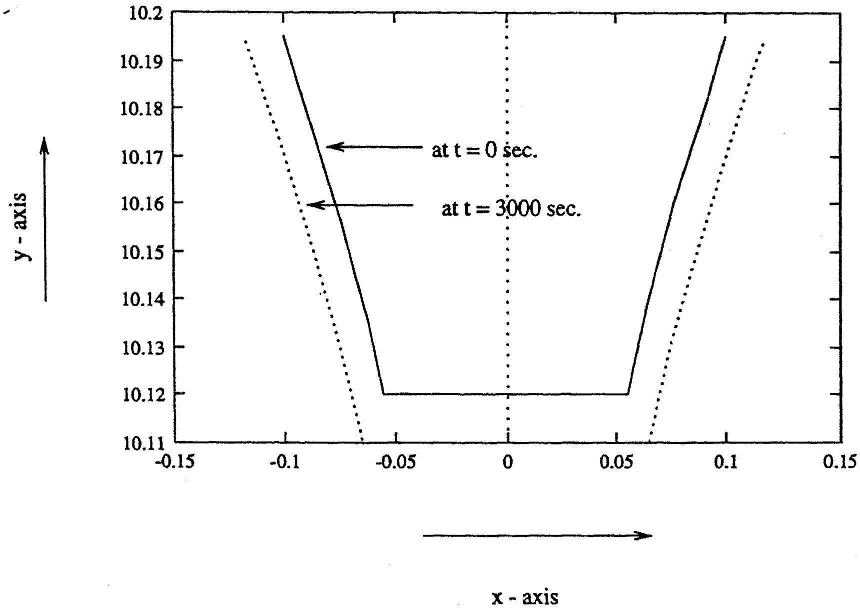


Figure 4.23: Temporal variation of cross section at inflow to Channel 1, Scenario-6,Procedure 1

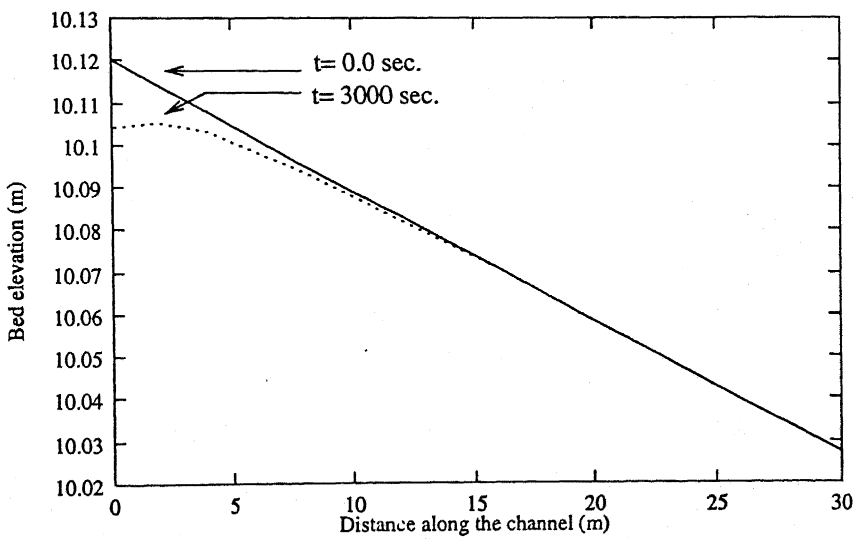


Figure 4.24: Bed profile along Channel 2, Scenario-6, Procedure 2

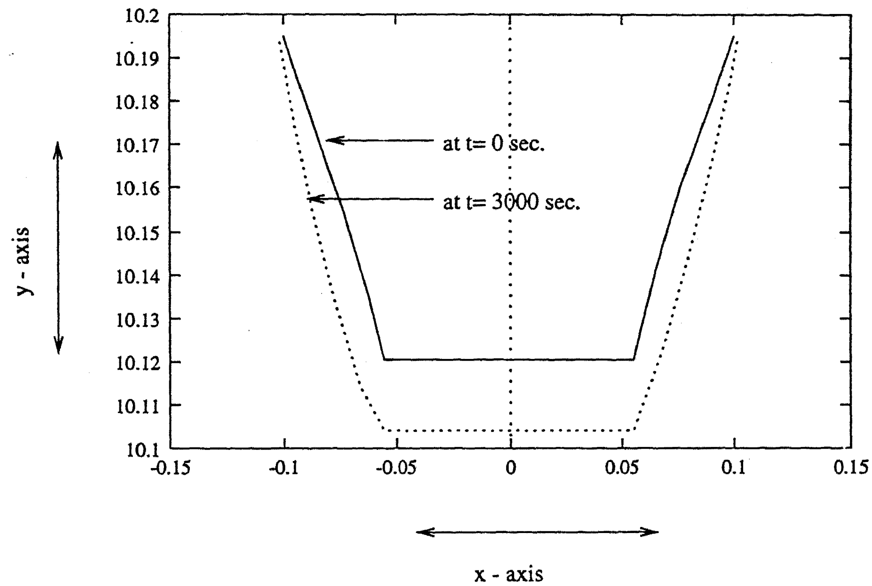


Figure 4.25: Temporal variation of cross section at inflow to Channel 1,Scenario-6,Procedure 2

channel i.e.,[2] when procedure 1 was adopted for modelling the changes in the cross-section. It is obvious from this figure that the degradation wave had not arrived at the junction point. Consequenty, no degradation occured (not shown here) in any other channel of the system. Figure 4.23 shows the change in the cross-section after 3000 seconds at the inflow section of the root channel. It can be clearly seen from Figure 4.23 that the procedure 1 for degradation has been correctly adopted in the computer code. Figures 4.24 and 4.25 show the change in the bed profile for the root channel and the cross-section at the inflow to the root channel after 3000 seconds when the procedure 2 was used to affect the change in the cross-section shape. A comparision of Figs. 4.22 and 4.24 indicates that the procedure adopted for proportioning the degradation across the cross-section affects the overall simulation results significantly. It can be observed that the bed wave had moved faster in the case of procedure 2 (Fig. 4.24). By proportioning the degraded area with respect to

the local shear stress, one gets more degradation at bottom. Consequently the bed slope for the reach changes much more than in the case of slope obtained by parallel degradation. Naturally these larger changes in the bed profile propagate faster. Once again it is emphasized here that the way to acount for change in the cross-section is a matter of engineering judgement. Conclusions regarding the correctness of a procedure should await more information from laboratory and field tests. The simulations presented here indicate only that the procedures are correctly encoded in the computer code.

Chapter 5

Summary And Recommendations For Furthur Investigations

This investigation presented a numerical model for the aggradation and degradation in tree-type channel networks. The model is based on a quasi-steady flow assumption and an uncoupled approach for the solution of the governing equations. The model can simulate the bed level variation in rectangular and non-rectangular channels.

The numerical model was verified by comparing the computed results with available experimental and other earlier numerical results. The numerical model simulated the bed level variation in the networks satisfactorily. However, The model did not give results when the Froude number was close to unity because the model solves the steady state energy equation for obtaining flow profiles. The model gave as good results as those obtained by a complete unsteady coupled model. The comparision between the simulated and measured aggradation in laboratory a flume was satisfactory. However, its applicability to natural river networks needs to be tested. The numerical model presented in this study is applicable for channels with beds composed of uniform meterial. This model dose not consider the bank stability.

The following is a list of subjects that require further research.

1. Mechanism of aggradation or degradation process at the confluence points in networks needs to be studied in more detail.

2. Two dimensional models need to be devoloped using quasi-steady technique.

3. Bed material in natural channels is seldom uniform and bed degradation is greatly controlled by armouring. Research needs to be done for inclusion of the effect armouring in models for networks.

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